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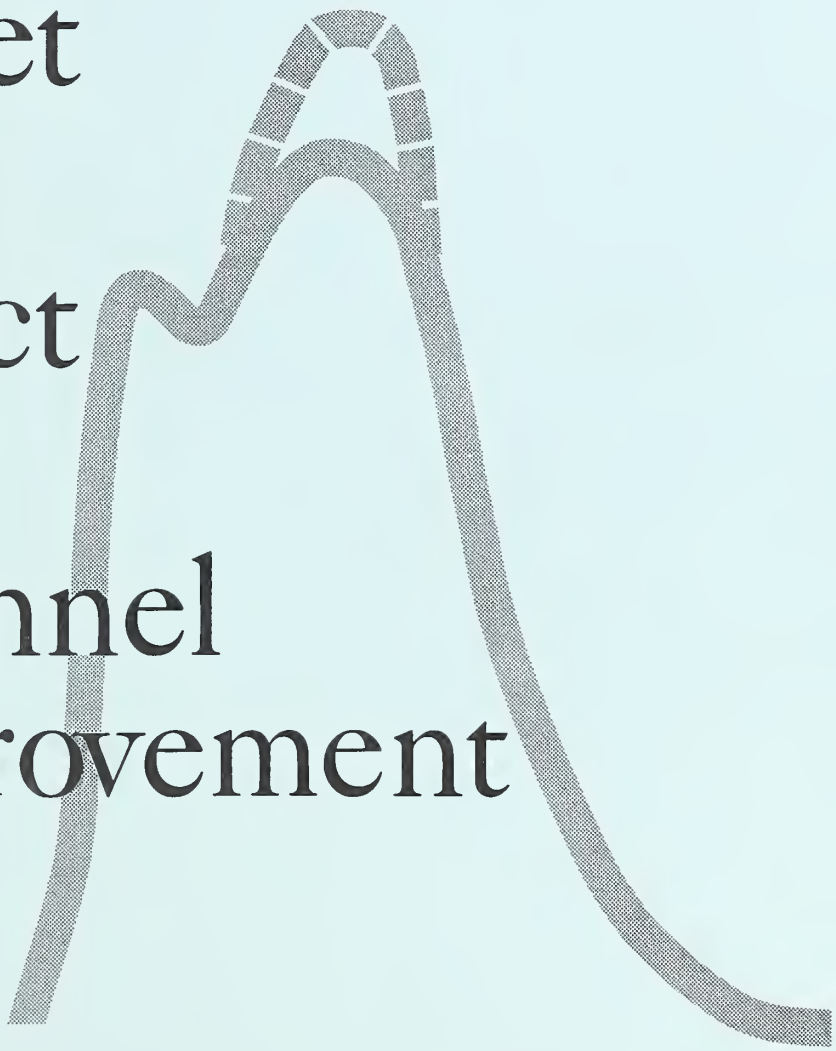

U. S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE
July 1963

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by

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Unit Hydrologist
Engineering and Watershed Planning Unit
Soil Conservation Service
Fort Worth, Texas

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LIST OF SYMBOLS

q_p = Peak discharge unit hydrograph

t_p = Lag time mid point unit rainfall duration t_r to peak of unit hydrograph, in hours

t_r = Unit rainfall duration

t_R = Unit rainfall duration adopted for study (2 hours)

L = Length of channel in miles

L_{ca} = Length of channel to center of watershed, in miles

S = Slope of channel in feet/mile

x = Routing coefficient

K = Travel time through reach

Synopsis

The storage required to offset the effect of increase in peak discharge caused by channel improvement in reaches below the termination of the improved channel, may be in two forms - floodwater retarding structures located on tributaries entering above the point where the channel improvement is terminated or channel storage which dissipates the peak (for all practical purposes) some distance downstream.

The Poteau River watershed P.L. 566 plan, recently approved for construction was selected for this study. This watershed is a contributor to the Poteau River and empties directly into the Wister Reservoir, built by the Corps of Engineers to control floods on the main stem of the Poteau River and to be operated as an integral part of the flood control system on the Arkansas River.

The analyses of improvements on this watershed were made using flood routing procedures shown in the National Engineering Handbook, Section 4, Hydrology, Supplement A, Chapter 17.

Records from a U. S. Weather Bureau recording rain gage located at Waldron, Arkansas and a U. S. Geological Survey stream gage located near Cauthron, Arkansas, were used to confirm results of flood routing procedures for present conditions.

Introduction:

The Soil Conservation Service policy in watershed planning and development is to apply flood reduction measures in the following order:

1. On-farm conservation practices.

2. Floodwater retarding structures.

3. Channel improvement and/or levees.

In a number of the watersheds conservation practices and floodwater retarding structures are sufficient to reach the objective of the local sponsoring agency.

The channel improvement planned in the Poteau River watershed was needed to reach two of the objectives.

1. Reduce the 1 percent chance flood to an acceptable level through the city of Waldron.

2. Reduce flooding through reaches M-3 and M-4 to a level where the agricultural potential of the flood plain could be developed.

Physical Data:

The Poteau River rises in west-central Arkansas and east-central Oklahoma above Wister Reservoir, a Corps of Engineers project. The river flows north below Wister Reservoir to its confluence with the Arkansas River at Fort Smith, Arkansas.

The Poteau River watershed includes that part of the Poteau River which heads in west-central Scott County, Arkansas, and flows in a westward direction for about 37 miles until it enters the flood pool of Wister Reservoir about four miles west of the Arkansas-Oklahoma State line. Of the 187,460 acres within this watershed, 176,130 acres are in Scott County, Arkansas and 11,330 acres are in LeFlore County, Oklahoma. The principal tributaries are East Fork, Jones, Cross, Square Rock, Shadley, and Loving Creeks.

The topography ranges from low mountainous, with elevations up to about 2,640 feet above sea level along the northern boundary, to a

gently rolling prairie-type landscape west of Waldron, where the general elevation is about 550 feet. The watershed lies in the Ouachita Highlands and Arkansas Valley Physiographic Regions. The major valleys and many of the minor valleys are developed along fault zones.

The watershed, roughly rectangular in shape, lies entirely within the Ouachita Highlands land resource area and is underlain by shales, siltstones, and sandstones of Pennsylvanian Age. Soils of the upland range from moderately deep, medium-texture, slowly permeable to shallow, medium-textured, and gravelly to stony. Most of the soils are in the latter category. The soils within the flood plain are fine silty to sandy loams.

Based on the 22-year gage record (1931-1952) at Waldron, Arkansas, the average annual rainfall is 46.36 inches. The maximum and minimum annual rainfall for the above period is 70.78 and 35.47 inches, respectively.

The mean rainfall by months, in inches, is as follows:

January	3.91	July	2.95
February	4.18	August	3.34
March	3.94	September	3.41
April	4.43	October	3.50
May	5.52	November	3.50
June	4.46	December	3.22

Mean temperatures range from 42.4 degrees Fahrenheit in January to 80.4 degrees in July. The minimum temperature of record is 20 degrees below zero and the maximum is 109 degrees above zero. The normal frost-free period of 222 days extends from March 29 to November 6.

Assumptions:

It was assumed that the following generally accepted theories were valid for this analysis.

1. Unit hydrograph theory as presented by L. K. Sherman.

2. The stage-storage curve approaches a straight line for each routing reach.
3. The dimensionless unit hydrograph shown as figure 3.16-3, National Engineering Handbook, Section 4, Supplement A, would produce suitable shaped unit graphs.

It was further assumed that a weighted velocity could be used to determine the routing coefficient required in the Improved Coefficient Routing Method shown in National Engineering Handbook, Section 4, Hydrology, Supplement A, Chapter 17.

Analysis:

Cross sections were run at representative locations on the main stem and tributaries. Data from these cross sections were processed through the IBM-650 (using programs developed by Paul Doubt, Head, Design Section, SCS) to obtain stage discharge curves and end area curves for present condition and with improved channel. Figure 1 compares the computed curve with a rating curve developed by U.S.G.S. using current meter measurements for the Cauthron gage.

An average velocity curve was developed for each cross section for present and with improved channel. Figures 2 and 3 show a typical cross section in routing reach M-3. A weighted velocity was computed using three points (50, 100 and 200 c.s.m.) as a basis. This weighted velocity was then used to compute the travel time through the segment of the routing reach represented by each cross section. The weighted velocity through each routing reach was also used to determine the coefficient "C"

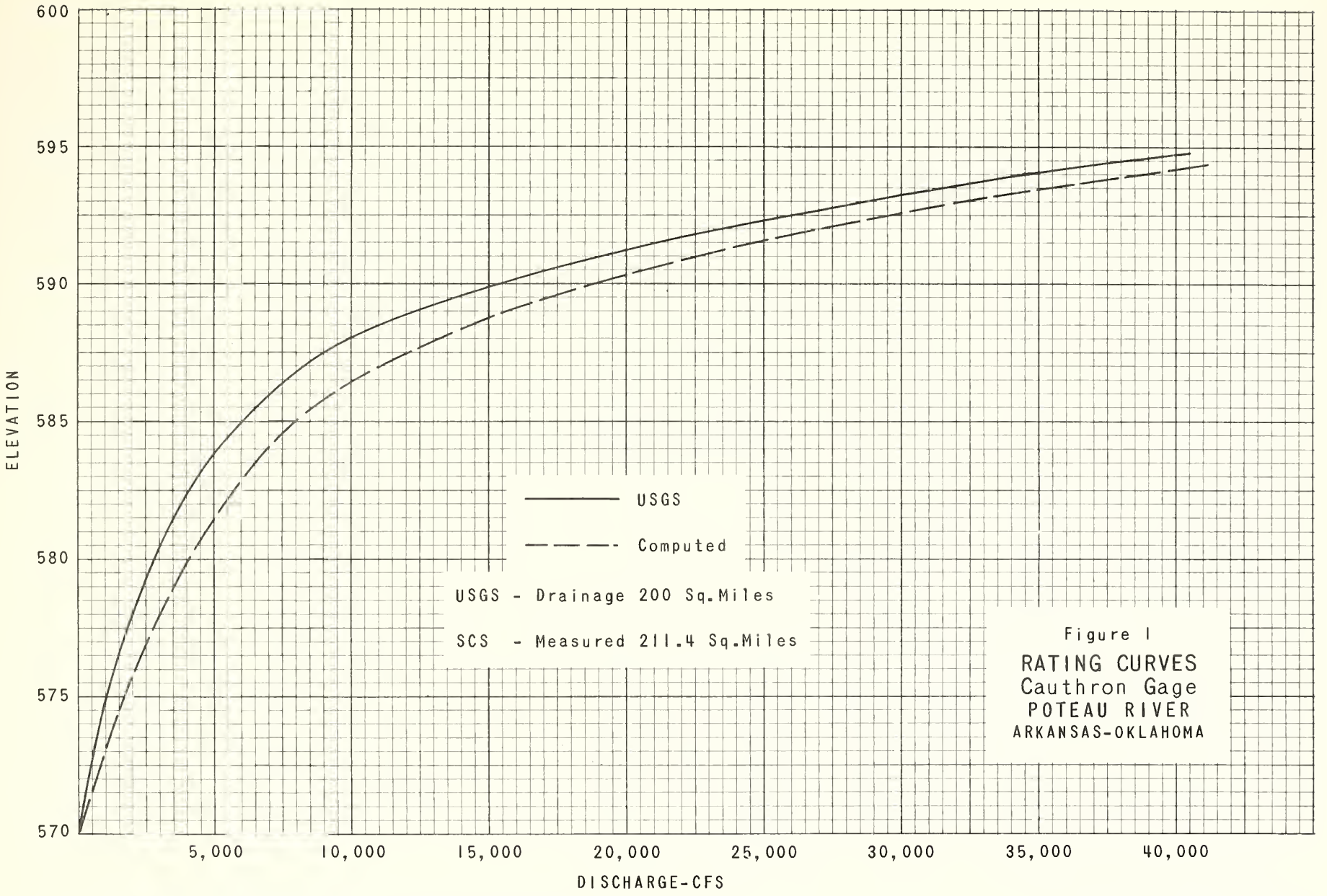

Figure 1
RATING CURVES
Cauthron Gage
POTEAU RIVER
ARKANSAS-OKLAHOMA

Figure 2

TYPICAL CROSS SECTION
WITH CHANNEL IMPROVEMENT ID-37
(In Routing Reach M-3)
POTEAU RIVER
ARKANSAS-OKLAHOMA

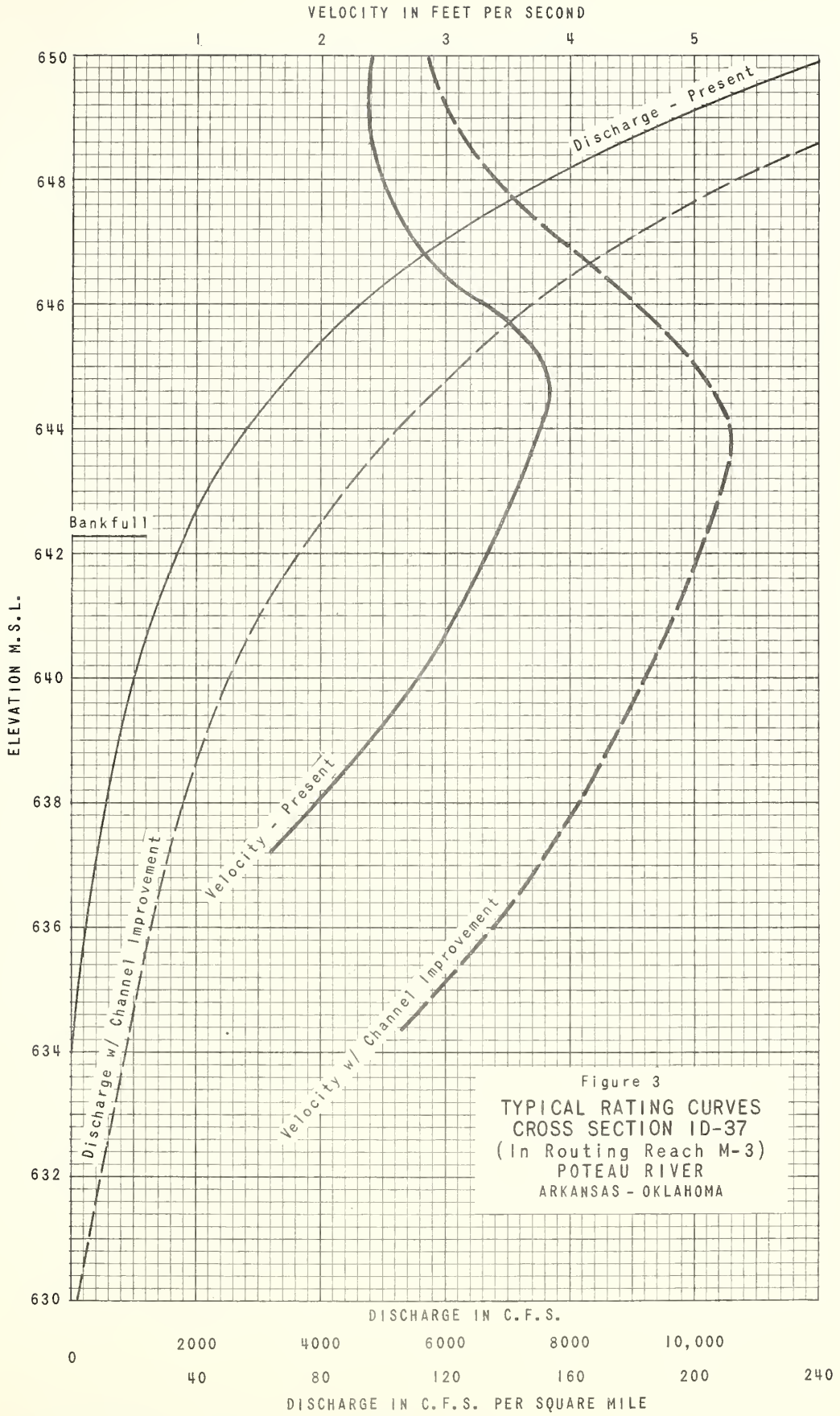

Figure 3

TYPICAL RATING CURVES
CROSS SECTION ID-37
(In Routing Reach M-3)
POTEAU RIVER
ARKANSAS - OKLAHOMA

(2x) used in routing (Figure 4 developed by Victor Mockus, Hydraulic Engineer, SCS, Washington, D. C.)

Beginning at the bottom of the watershed routing reach lengths were selected, and minor adjustments made, that would permit the use of uniform Δts that would make $\Delta t = 2Kx$, which is a requirement in the Improved Coefficient method of routing. Some variations from this procedure of selecting routing reach length was necessary due to the drainage pattern and to better show the effects of the proposed structures and channel improvement. Pertinent data for each routing reach selected are shown in table 1.

Data from three stream gages on Six Mile Creek in Arkansas and the gage located near Cauthron on Poteau River watershed were analyzed and the results are shown on figure 5. This figure was then used to determine the time to peak of the unit hydrograph for various segments of the watershed. This time to peak was adjusted (figure 6) to bring all of the hydrographs to a 2-hour unit hydrograph. Having the adjusted time to peak, the formula $q = \frac{484A}{t_p}$ (developed by Victor Mockus, National Engineering Handbook, Section 4, Hydrology, Supplement A) was used to compute the peak of the 2-hour unit hydrograph.

Two-hour unit hydrographs were developed using the computed peak discharge and the dimensionless unit hydrograph (figure 3.16-3, NEH-4, Supplement A) for each segment of the watershed (table 2).

An example is given on the following page.

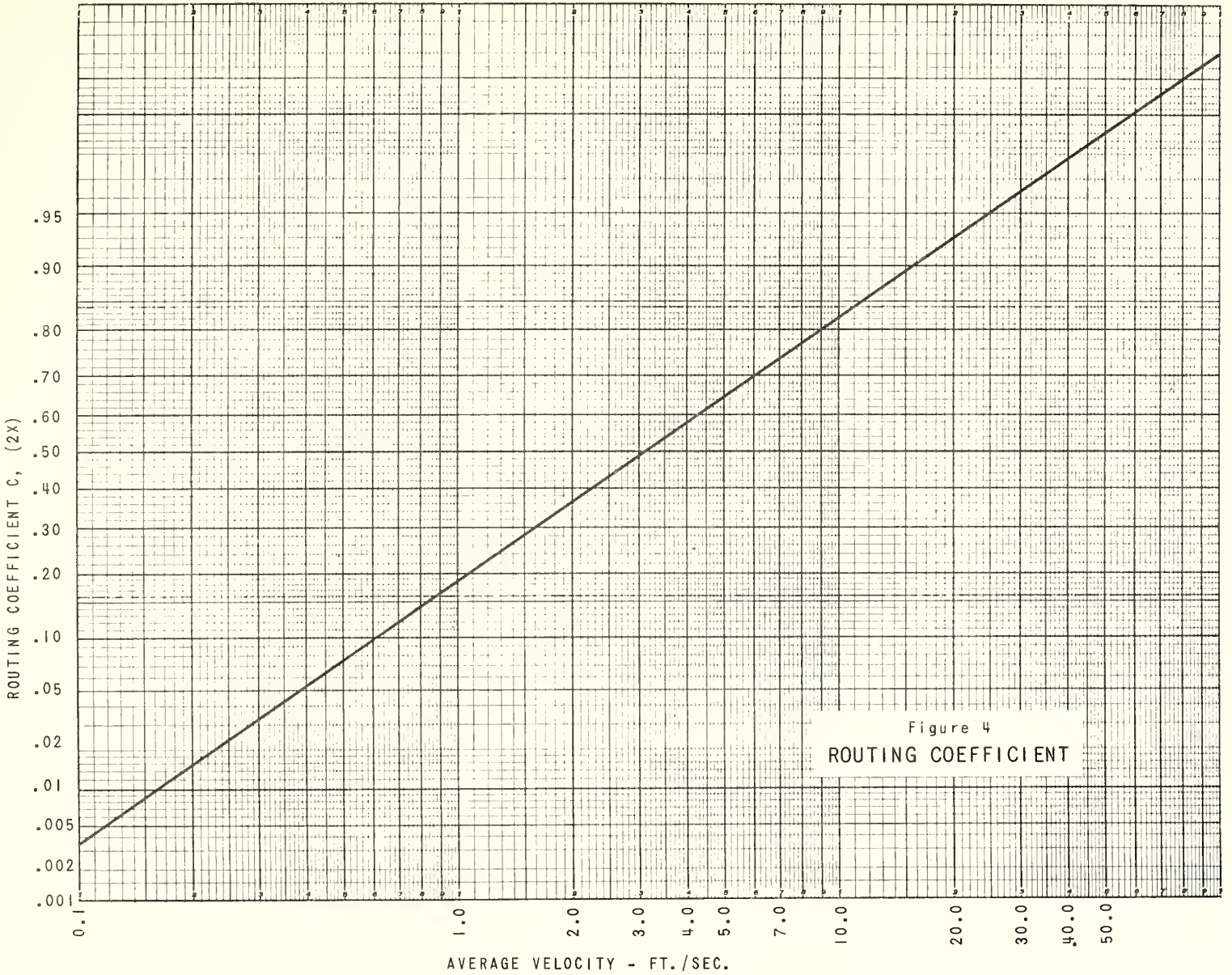

Figure 4

ROUTING COEFFICIENT

Developed by Victor Mockus, Hydraulic Engineer, SCS, Washington, D.C.

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TABLE 1

ROUTING FACTORS BY REACHES

Reach	Condition	From	To	K	V	2x	△t
M-1	Present	Site 1	ID-48	2.00	3.0	.50	1.0
M-2	Present	ID-48	ID-41	2.37	2.4	.42	1.0
	With Channel			1.90	3.1	.53	1.0
M-3	Present	ID-41	ID-35	2.20	1.7	.34	0.75
	With Channel			1.20	3.0	.50	0.60
M-4	Present	ID-35A	ID-30	3.77	1.6	.27	1.0
	With Channel 1/			2.50	2.4	.40	1.0
				2.00	3.0	.50	1.0
M-5	Present 1/	ID-30	ID-28	1.58	1.8	.33	0.5
				0.95	3.0	.52	0.5
M-6	Present 1/	ID-28	ID-23	2.35	2.8	.43	1.0
				2.00	3.0	.50	1.0
M-7	Present 1/	ID-23	ID-19	2.35	2.8	.43	1.0
				2.00	3.0	.50	1.0
M-8	Present	ID-19	ID-12	2.84	2.0	.35	1.0
M-9	Present	ID-12	ID-9	2.84	2.0	.35	1.0
M-10	Present	ID-9	ID-1	5.00	2.3	.40	2.0
E-1	Present 1/	ID-8	ID-1	3.21	1.8	.33	1.0
				1.93	3.1	.52	1.0
S-2	Present 1/	ID-7	ID-1	3.82	1.3	.25	1.0
				2.00	3.0	.50	1.0
L-1	Present	ID-7	Main Stem	.75	5.4	.66	0.5
T-1	Present	Site 8	Main Stem	2.50	2.2	.40	1.0
R-1 2/	Present 1/	Site 19	Main Stem	6.50	1.9	.35	2.25
				4.20	3.0	.50	2.10
J-1	Present	Site	ID-6	1.35	3.4	.52	0.75
J-2	Present	ID-6	Main Stem	1.35	3.4	.52	0.75
H-1	Present	Site 7	Main Stem	1.00	3.0	.50	0.50
WS-1	Present	Site	Main Stem	.48	3.4	.52	0.25

1/ All channels above gage that had a weighted average velocity of less than 3.0 ft./sec. were assumed to be improved to obtain a weighted average velocity of 3.0 ft./sec.

2/ Routing through this reach was performed using $\Delta t = 0.75$ present and 0.70 with channel. Three routings were made in order to conform to limit of $\Delta t \leq \frac{Tp}{5}$ and to be able to add in sites 18 and 13 at the proper time.

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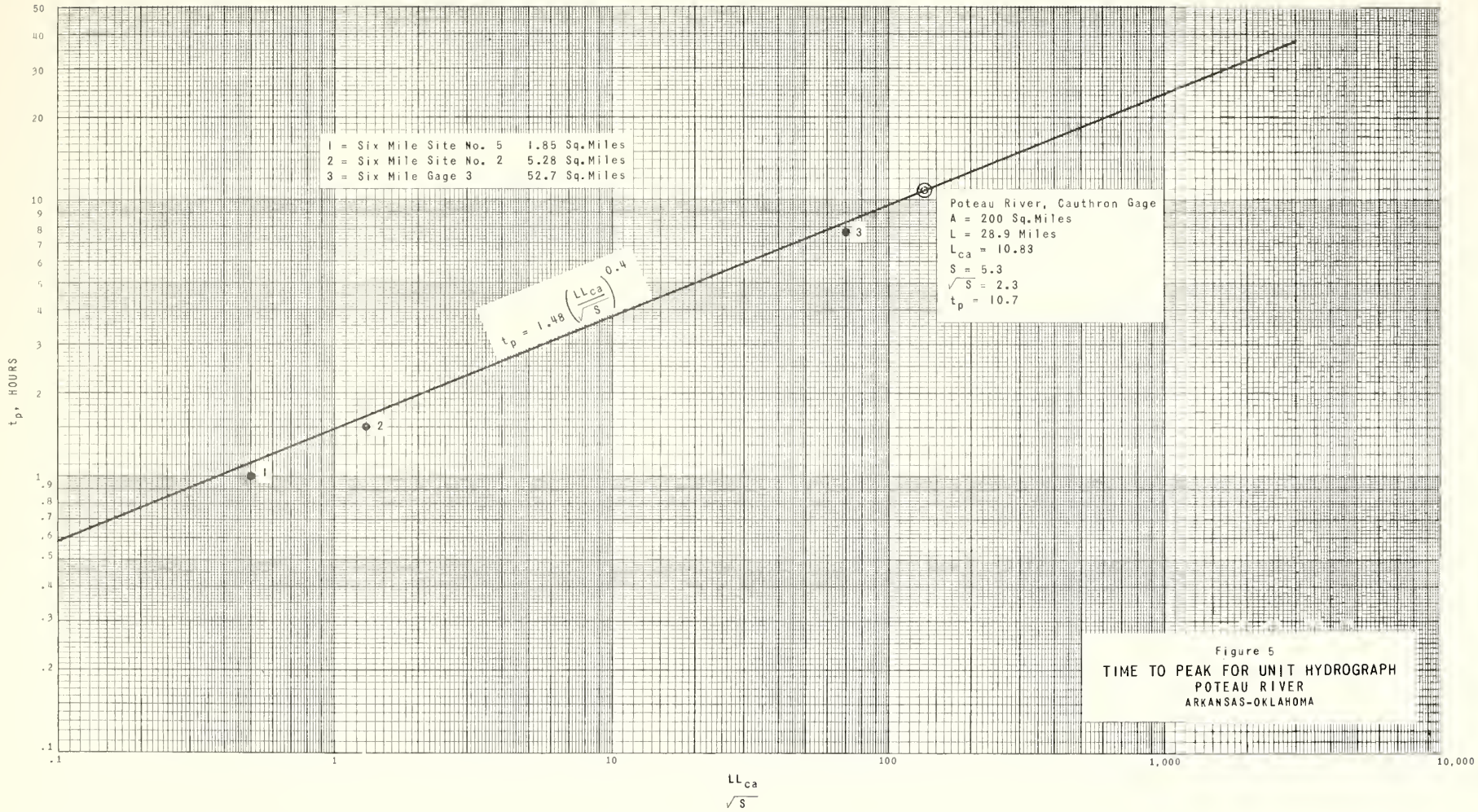

Figure 5
TIME TO PEAK FOR UNIT HYDROGRAPH
POTEAU RIVER
ARKANSAS-OKLAHOMA

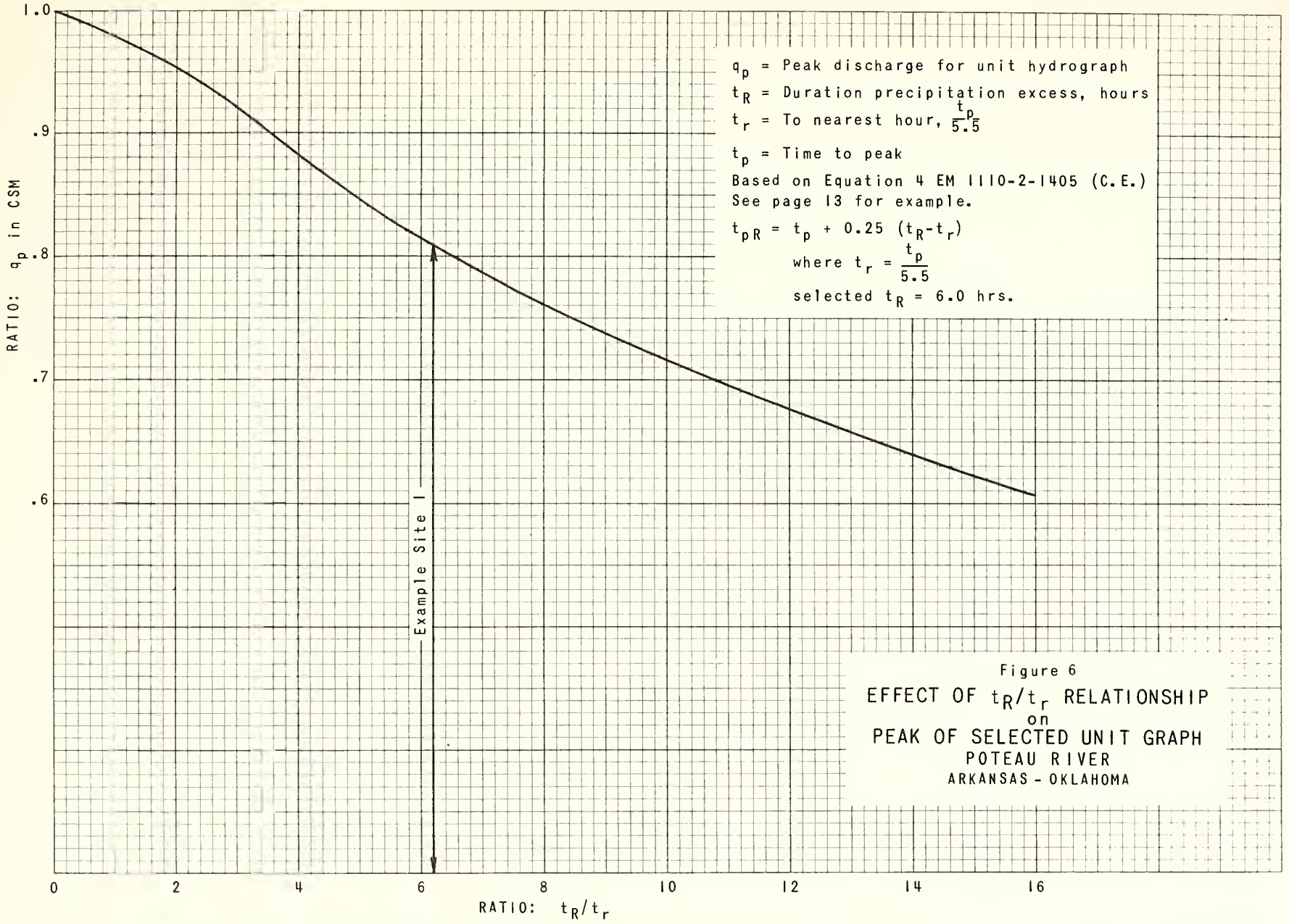

Figure 6

EFFECT OF t_R/t_r RELATIONSHIP
on
PEAK OF SELECTED UNIT GRAPH
POTEAU RIVER
ARKANSAS - OKLAHOMA

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TABLE 2

UNIT HYDROGRAPH
Data

Site or Area	Drainage Area Sq.Mi.	L Miles	L_{ca} Miles	s Ft./Mile	$\sqrt{s}$	$\frac{LL_{ca}}{\sqrt{s}}$	t_p hours	$q = \frac{484A}{t_p}$	Adjusted to 2 hr. t_p	Unit Graph q
Site 1	3.20	5.55	2.10	51.8	7.17	1.62	1.8	860	2.2	710
M-1	8.51	6.00	2.86	22.7	4.77	3.61	2.5	1,650	2.9	1,420
M-2	6.28	4.20	1.97	13.2	3.64	2.27	2.1	1,450	2.5	1,220
Site 5	23.0	19.0	6.13	21.7	4.66	24.4	5.4	2,060	5.8	1,920
E-1	6.64	5.44	3.58	8.45	2.90	6.70	3.2	1,000	3.5	920
M-3	6.67	3.86	1.92	5.28	2.30	3.22	2.4	1,340	2.8	1,150
Site 6	7.20	10.9	4.50	60.7	7.79	6.28	3.1	1,120	3.5	990
S-1	1.34	3.32	1.00	25.9	5.09	0.65	1.2	540	1.6	410
S-2	8.00	4.85	3.16	9.00	3.00	5.12	2.9	1,330	3.3	1,170
M-4	6.64	5.90	2.73	2.64	1.82	8.85	3.6	900	3.9	830
Site 19	11.95	13.0	4.68	18.0	4.24	14.90	4.4	1,310	4.7	1,230
Site 18	3.58	4.34	1.61	78.2	8.85	0.80	1.4	1,230	1.8	960
Site 13	4.96	4.34	1.40	64.5	8.04	0.76	1.3	1,850	1.7	1,420
R-1	10.29	10.4	4.87	9.00	3.00	16.9	4.6	1,080	4.9	1,020
Site 17	3.12	5.00	2.90	40.2	6.34	2.29	2.1	720	2.5	600
Site 15	34.40	23.3	10.3	19.55	4.43	54.1	7.4	2,250	7.7	2,170
J-1	7.32	3.92	2.08	10.6	3.16	2.58	2.2	1,610	2.6	1,360
J-2	4.23	2.98	1.69	5.80	2.40	2.10	2.0	1,020	2.4	850
HP-1	13.85	8.80	3.04	14.25	3.77	7.10	3.3	2,070	3.6	1,900
Site 7	1.42	2.44	1.50	143.0	12.0	0.31	0.9	760	1.3	530
M-5	5.59	4.65	2.73	37.0	6.08	2.09	2.0	1,350	2.4	1,130
Site 8	3.44	5.07	1.35	88.3	9.39	0.73	1.3	1,280	1.7	980
M-6	10.89	6.28	2.18	2.12	1.46	9.38	3.7	1,430	4.0	1,320
Site 16	7.92	5.75	2.57	90.4	9.50	1.56	1.8	2,130	2.2	1,920
M-7	14.98	6.20	2.65	2.64	1.62	10.20	3.8	1,910	4.1	1,760
Site 9	3.63	3.61	1.61	155.0	12.45	0.47	1.1	1,580	1.5	1,160
M-8	16.50	8.08	4.40	3.17	1.78	20.00	5.0	1,600	5.3	1,510
Site 10	9.53	4.21	2.46	87.7	9.37	1.11	1.5	3,070	1.9	2,420
M-9	18.90	5.60	2.75	2.64	1.62	9.50	3.7	2,480	4.0	2,290
M-10	28.29	9.45	5.18	6.35	2.52	19.3	4.9	2,790	5.2	2,620

Example:

Poteau River at Cauthron gage, Drainage Area = 200 square miles

See figure 5 $t_p = 10.7$ hours

Therefore $t_R = 10.7$

$$t_r = \frac{10.7}{5.5} = 1.95 \text{ (unit duration)}$$

Use 2-hour unit graph

Using 2-hour unit duration for Site 1

DA = 3.20, L = 5.55, L_{ca} = 2.10, S = 51.8, $\sqrt{S}$ = 7.17

$\frac{LL_{ca}}{\sqrt{S}} = 1.62$ $t_p = 1.8$ (figure 5)

$\frac{1.80}{5.5} = .328$

Ratio of $\frac{2.0}{.328} = 6.1$

Enter figure 2 with 6.1 Read 0.81

Adjusted $t_p = \frac{1.8}{.81} = 2.2$ hours

Adjusted $q = \frac{484 \times 3.20}{2.2} = 710$ cfs

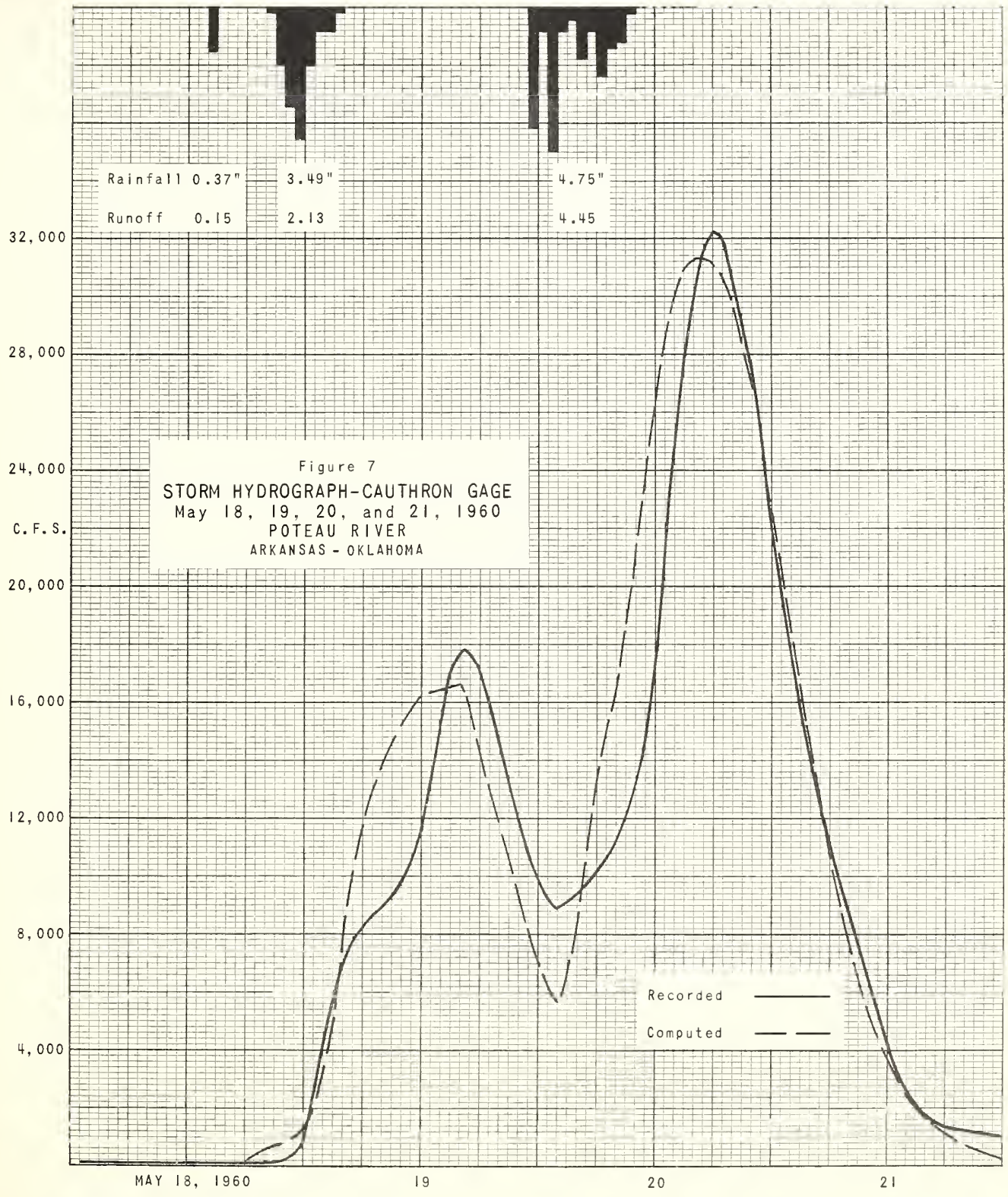
Flood routings were performed for the following conditions:

1. Present.
2. With planned floodwater retarding structures.
3. With floodwater retarding structures and 10.1 miles of channel improvement (as planned).
4. With 10. 1 miles of channel improvement only, on the main stem.
5. With 27 miles of channel improvement assumed on the main stem above the Cauthron gage.

6. With 27 miles of channel improvement assumed on the main stem above the Cauthron gage and planned floodwater retarding structures.

The routing of unit hydrographs for present condition was completed down to the Cauthron gage. At this point four floods, varying in size from the maximum of record to the 80 percent chance, were selected to check the accuracy of the unit hydrograph developed from flood routing for present condition at the Cauthron gage. The volume of runoff as measured at the gage was broken into 2-hour segments in proportion to the 2-hour amounts of rainfall (U.S.W.B. Recording Gage, Waldron). These two-hour amounts of runoff were then applied to the unit hydrograph to reproduce flood hydrographs, which compared favorably with the actual flood hydrographs of the four storms. Figures 7 through 10 substantiate the assumption that a weighted velocity can be used to determine the routing coefficient, (used in the Improved Coefficient Routing Method) and that a unit hydrograph developed by this routing can be used to reproduce flood hydrographs for various sized floods. The percent chance of these storms is based on runoff rather than rainfall. The antecedent moisture condition was between II and III for each of the four storms analyzed. Figures 11 through 14 show that the rainfall recorded at Waldron (U.S.W.B. Gage) for the four storms was equal, for all practical purposes, to the weighted rainfall on the watershed above the Cauthron gage.

The flood routing was completed for present condition and with planned floodwater retarding structures assumed in place. These two routings were used as a base for computing the increase in peak


Rainfall 0.37"
3.49"
4.75"
Runoff 0.15
2.13
4.45
32,000
28,000
24,000
C.F.S.
20,000
16,000
12,000
8,000
4,000
Figure 7
STORM HYDROGRAPH-CAUTHRON GAGE
May 18, 19, 20, and 21, 1960
POTEAU RIVER
ARKANSAS - OKLAHOMA
Recorded
Computed
MAY 18, 1960
19
20
21

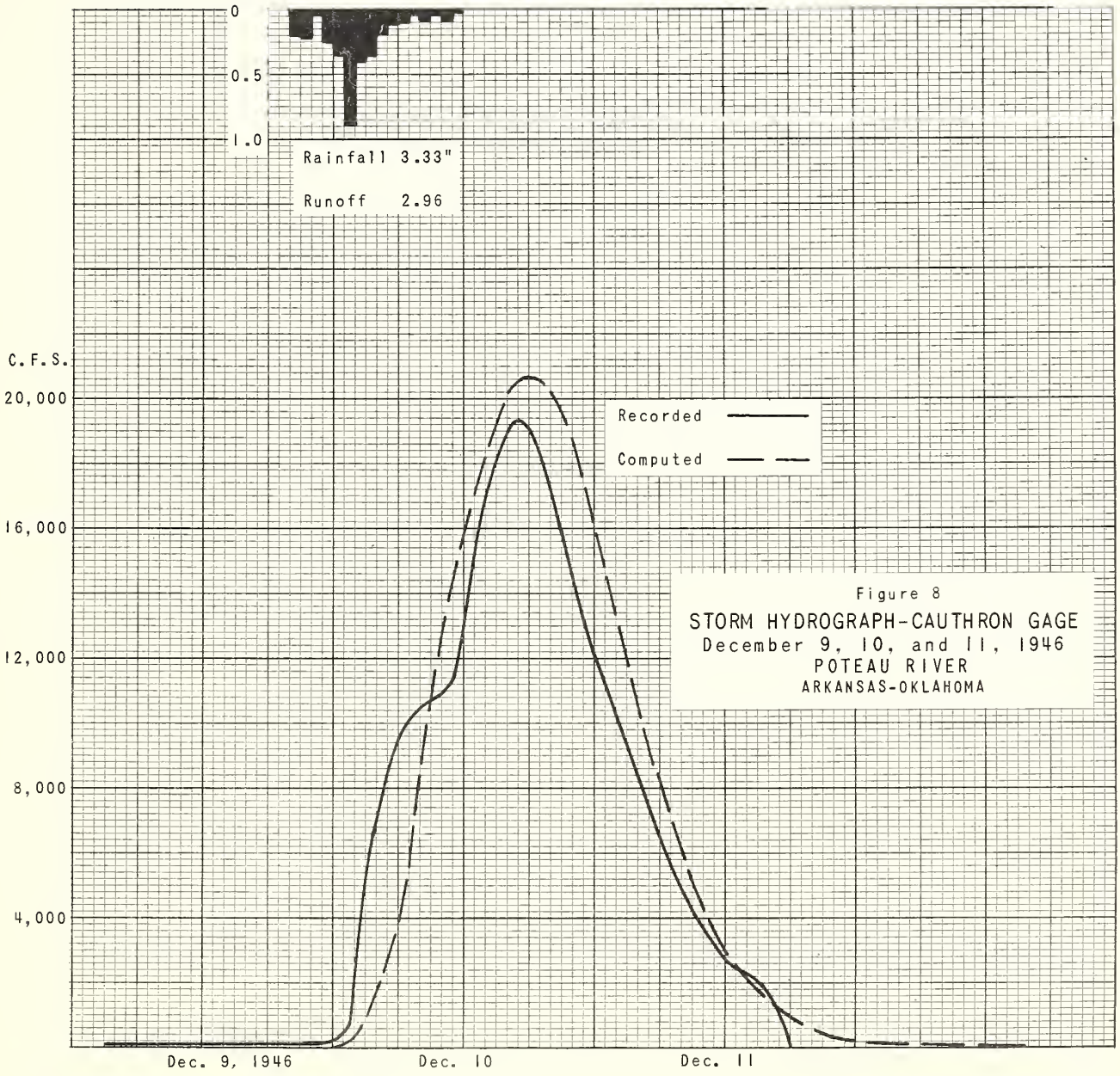


Figure 8

STORM HYDROGRAPH-CAUTHRON GAGE
December 9, 10, and 11, 1946
POTEAU RIVER
ARKANSAS-OKLAHOMA

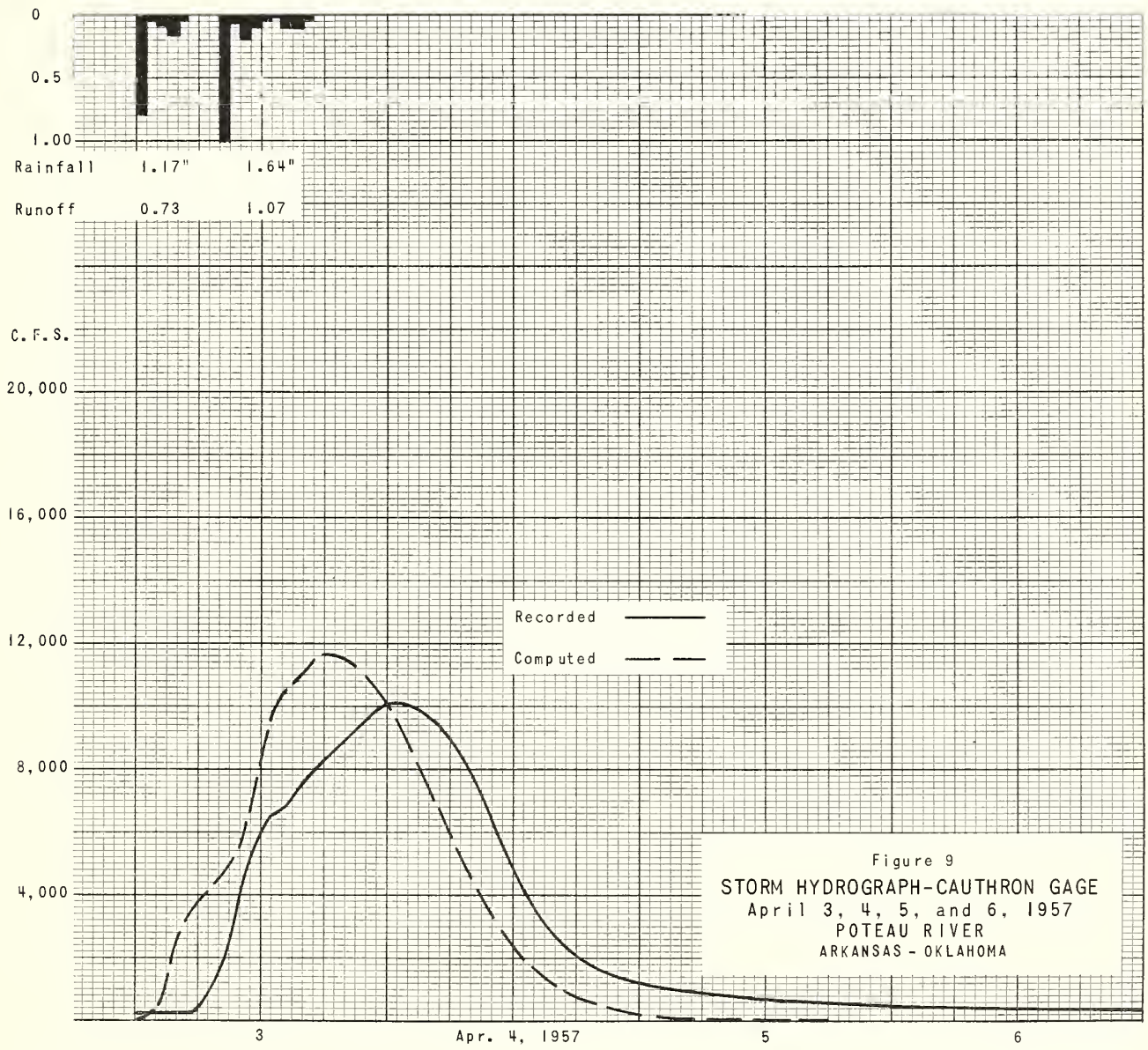


Figure 9

STORM HYDROGRAPH-CAUTHRON GAGE
April 3, 4, 5, and 6, 1957
POTEAU RIVER
ARKANSAS - OKLAHOMA

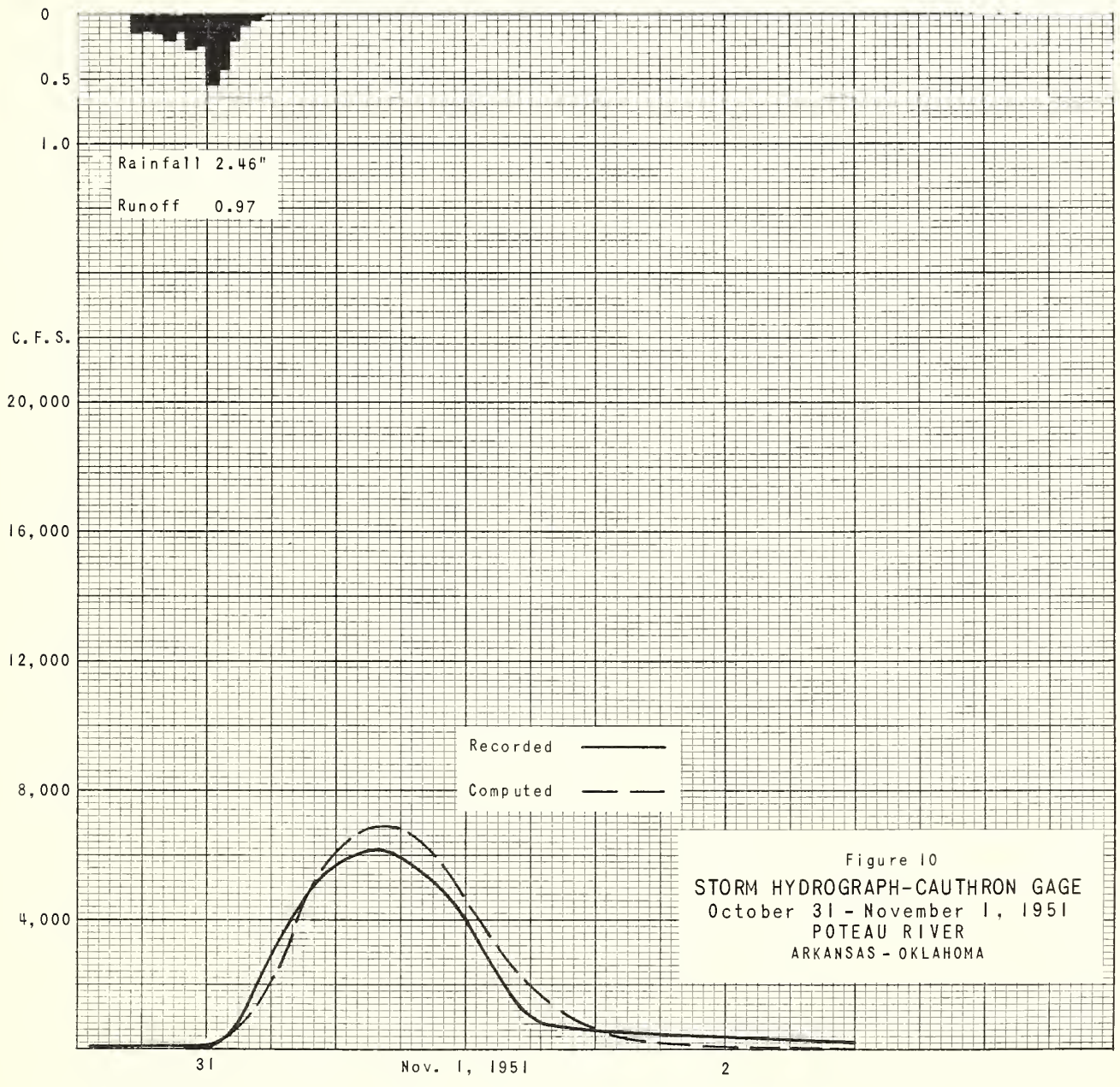


Figure 10

STORM HYDROGRAPH-CAUTHRON GAGE
October 31 - November 1, 1951
POTEAU RIVER
ARKANSAS - OKLAHOMA

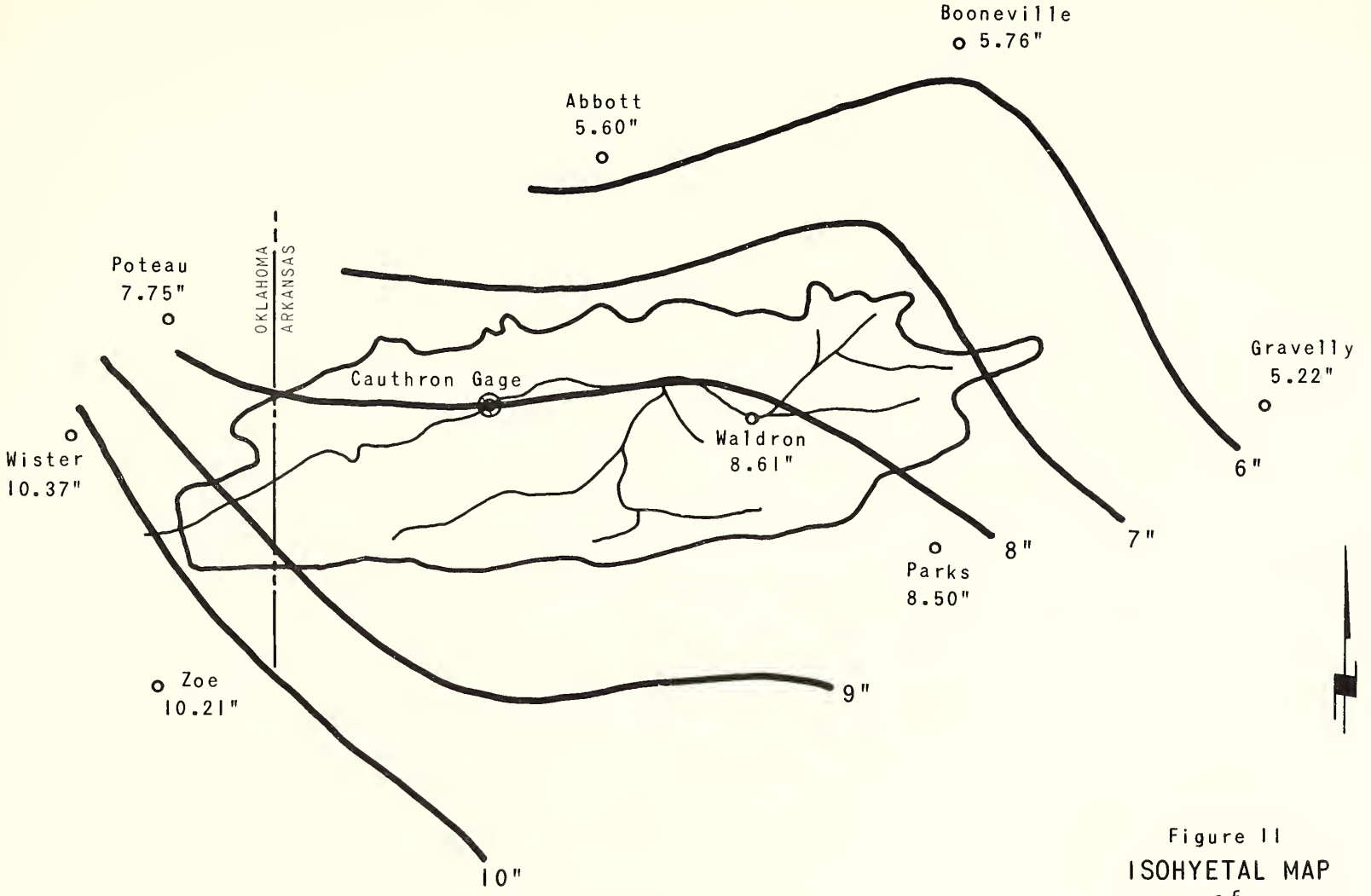


Figure 11

ISOHYETAL MAP
of
RAINFALL
May 18-19-20, 1960
POTEAU RIVER
ARKANSAS-OKLAHOMA

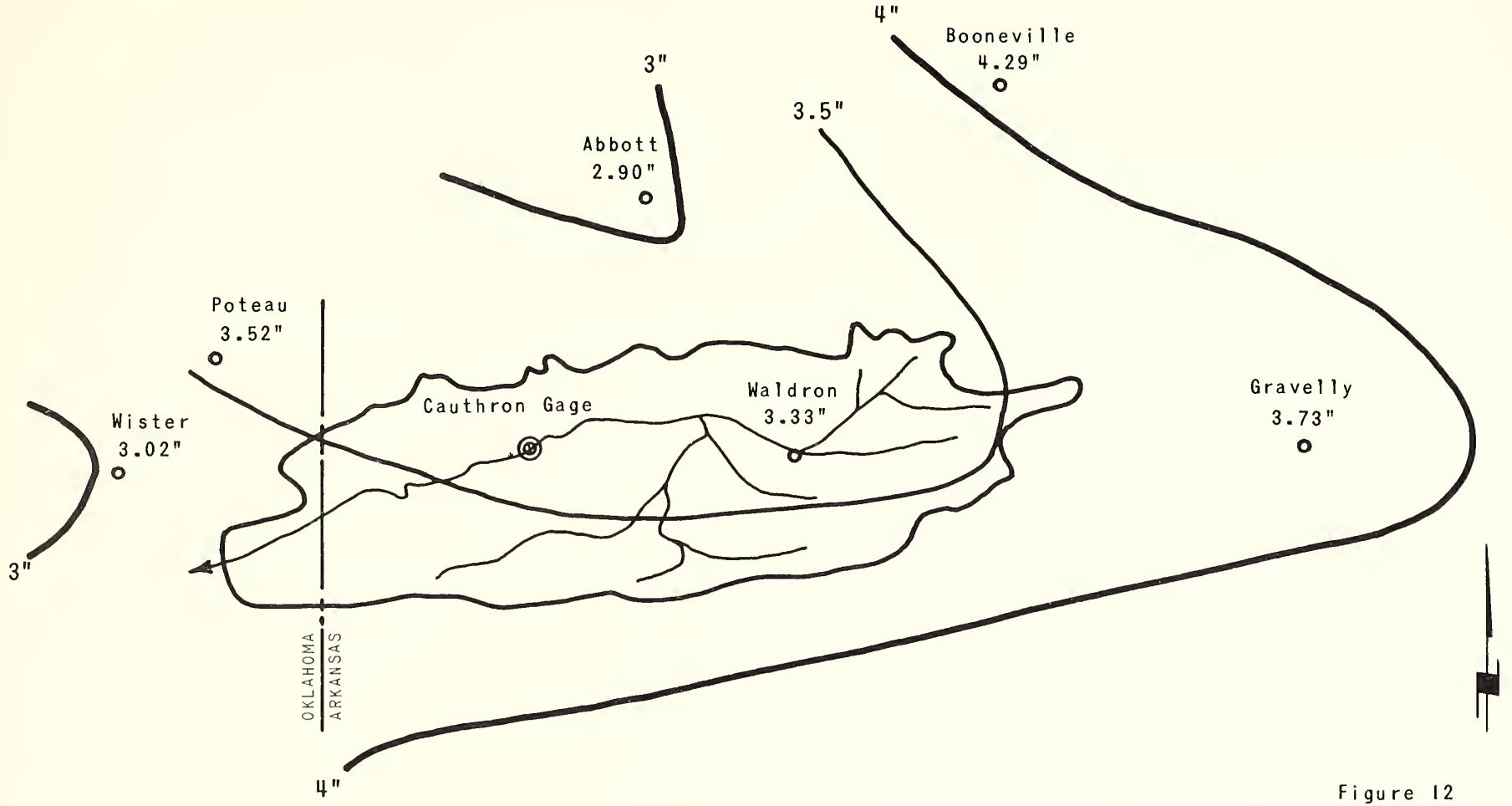


Figure 12
ISOHYETAL MAP
of
RAINFALL
Dec. 9-10, 1946
POTEAU RIVER
ARKANSAS-OKLAHOMA

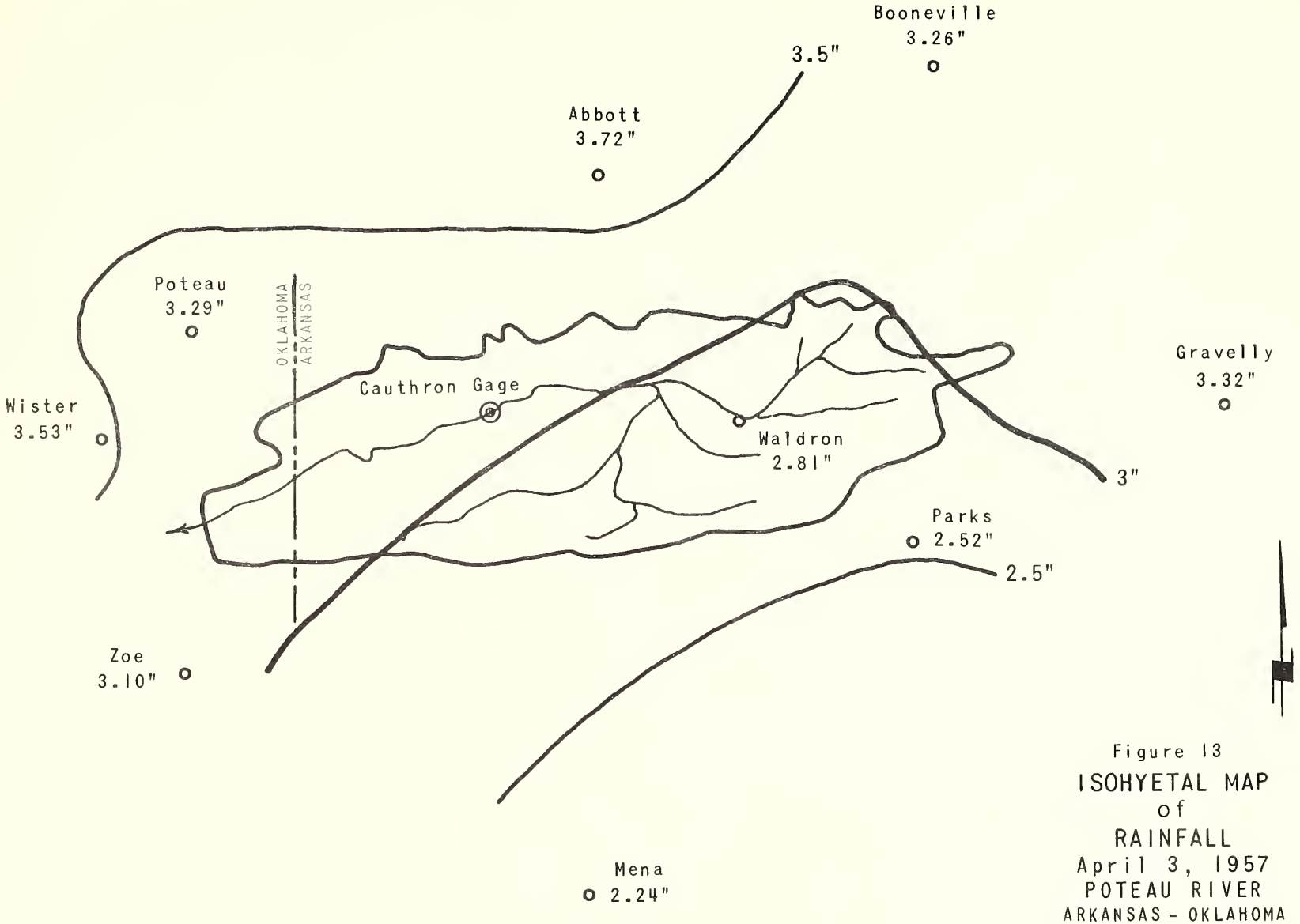


Figure 13
ISOHYETAL MAP
of
RAINFALL
April 3, 1957
POTEAU RIVER
ARKANSAS - OKLAHOMA

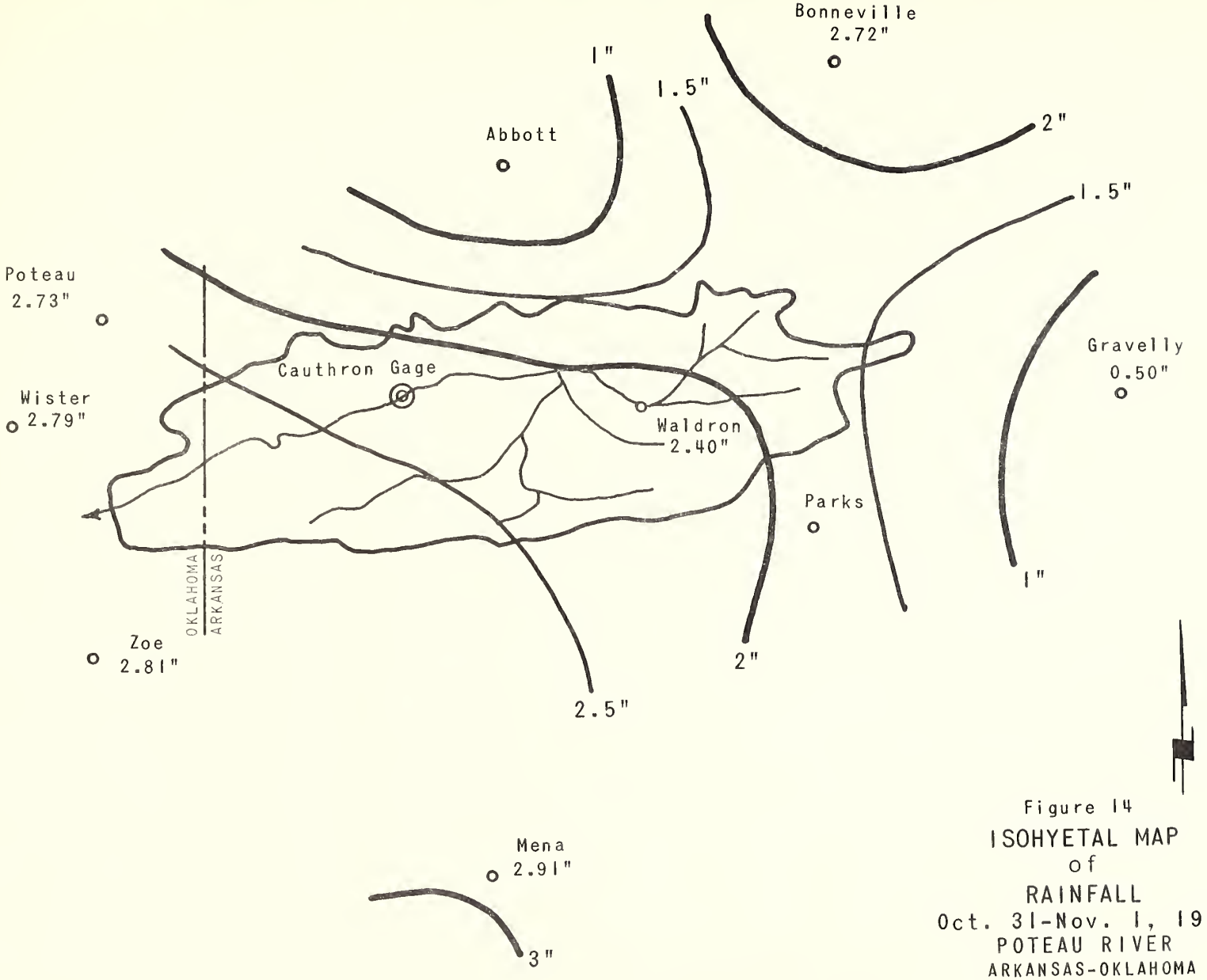


Figure 14
ISOHYETAL MAP
of
RAINFALL
Oct. 31-Nov. 1, 1951
POTEAU RIVER
ARKANSAS-OKLAHOMA

discharge (table 3) and decrease or increase in stage caused by channel improvement (table 4).

The plan as developed shows 10.1 miles of channel improvement in routing reaches M-2, M-3, and M-4. The channel through this area was enlarged and minor changes in alignment were made. The weighted velocity through these reaches was increased approximately 50 percent. New routing coefficients were developed for the reaches through which the channel was improved and two more flooding routings made (one with 10.1 miles of channel improvement) using the same unit hydrographs for incremental areas that were used in the first two routings. This completed the analysis of the watershed as planned.

In order to show what might happen on a watershed it was assumed that the channel was improved through routing reaches M-5, M-6, M-7 on the main stem, and E-1, S-2, and R-1 on the tributaries. The extent of this channel improvement was to bring the weighted velocity up to approximately 3 feet/sec. It was also assumed that the channel improvement below routing reach M-5 would increase the weighted velocity in routing reach M-4 to 3 feet/sec.

New routing coefficients were developed for the reaches through which the channel was assumed to be improved and two more flood routings made.

1. With 27 miles of channel improvement on the main stem.
2. With structure and 27 miles of channel improvement on the main stem.

TABLE 3

MAIN STEM ROUTING - UNIT HYDROGRAPH

Location	Reach No.	Drainage Area Sq.Mi.	Present q	Present T_p	Planned Channel q	Planned Channel T_p	Percent Increase	Assumed Channel q	Assumed Channel T_p	Percent Increase	Drainage Area Sq.Mi.	Structures Only q	Structures Only T_p	As Planned Structures and Channels q	As Planned Structures and Channels T_p	Percent Increase	Assumed Structures and Channels q	Assumed Structures and Channels T_p	Percent Increase
Head	M-1	3.20	710	2.2	710	2.2	0	710	2.2	0	0	Release discharge only.							
Head	M-2	11.71	1,783	3.5	1,783	3.5	0	1,783	3.5	0	8.51	1,420	3.0	1,420	3.0	0	1,420	3.0	0
Head	M-3	47.63	3,128	5.25	3,368	4.8	7.5	3,624	4.8	16.0	21.43	2,430	3.75	2,620	3.6	8.0	2,620	3.6	8.0
Head	M-4	70.84	4,042	6.0	4,684	5.0	16.0	5,263	5.0	30.0	37.44	3,597	4.0	4,332	4.0	20.0	4,396	4.0	22.5
Head	M-5	171.18	7,910	8.0	8,795	7.0	11.0	9,838	6.5	24.0	79.77	5,786	4.5	5,855	4.5	18.5	7,306	4.5	26.5
Head	M-6	181.63	7,806	10.0	8,577	9.0	10.0	9,793	7.0	25.0	85.36	5,410	6.0	6,345	6.0	17.1	7,143	6.0	32.0
Head	M-7	200.44	7,693	12.0	8,342	11.0	8.6	9,681	9.0	26.0	96.25	5,262	8.0	6,040	8.0	14.7	7,011	7.0	33.0
Head 1/	M-8	215.42	7,590	14.0	8,169	13.0	8.0	9,583	11.0	26.2	111.23	5,199	10.0	5,863	10.0	13.0	6,924	9.0	33.5
Head	M-9	245.08	7,454	16.0	7,947	15.0	6.7	9,203	13.0	23.6	127.35	5,131	12.0	5,661	12.0	10.5	6,527	11.0	27.0
Head	M-10	263.98	7,271	18.0	7,750	18.0	6.2	8,823	16.0	21.0	146.63	4,956	14.0	5,390	14.0	8.7	6,119	14.0	23.0
Foot	M-10	292.27	6,889	22.0	7,226	22.0	5.0	8,090	20.0	17.5	174.92	4,631	18.0	4,970	18.0	7.4	5,467	18.0	18.0

1/ Check on Cauthron Gage.

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TABLE 4

CHANGE IN STAGE
(Feet)

Head Reach	Compared to Present						Compared to Structures Only					
	Present with 10.1 Mi. Channel			Present with 27 Mi. Channel			Structures with 10.1 Mi. Channel			Structures with 27 Mi. Channel		
	Unit Hydro-graph	Flood of Record	Annual Flood	Unit Hydro-graph	Flood of Record	Annual Flood	Unit Hydro-graph	Flood of Record	Annual Flood	Unit Hydro-graph	Flood of Record	Annual Flood
M-3	-1.4	-0.8	-1.8	-	-	-	-1.6	-1.6	-2.4	-	-	-
M-5	+0.6	+0.2	+0.4	-	-	-	+0.5	+0.2	+1.0	-	-	-
M-8	+0.4	+0.2	+0.6	+1.4	+0.8	+1.2	+0.8	+0.1	+0.6	+2.0	+0.3	+1.5
Bottom M-10	+0.8	+0.3	+0.8	+1.1	+0.7	+1.3	+0.5	+0.1	+0.5	+1.5	+0.4	+1.0

Unit hydrographs were developed by flood routing for each condition. Figures 15 through 18 show the changes in the peak discharge of the unit hydrograph caused by the various structural measures at strategic points along the stream.

Using the unit hydrographs developed for various conditions by flood routing, flood hydrographs were developed for the flood of record and the 80 percent chance flood. Figures 19 through 24 show the effect of various structural measures or the peak discharge of the flood of record, and a smaller flood (approximately 80 percent chance).

Findings and Conclusions:

In order to locate storage (flood or channel) to offset the increase in peak discharge caused by channel improvement, two studies were made.

The first study was to route the unit hydrograph from area HP-1 downstream (for both present condition and with 27 miles of channel improvement) to determine the effect of a floodwater retarding structure that would control the runoff from this area (figure 25).

This unit hydrograph, when placed in the proper time sequence at the head of reaches M-5, M-8 and the foot of the watershed, shows that a structure controlling area HP-1 would offset the effect of the 10.1 miles of improved channel as planned. However, it would not offset the effect of the 27 miles of improved channel assumed in this study.

The second study was to route the unit hydrograph for each of the six conditions through reach M-10 (assuming reach M-10 to be a representative reach) several times to show the rate of dissipation of the increased peak caused by channel improvement.

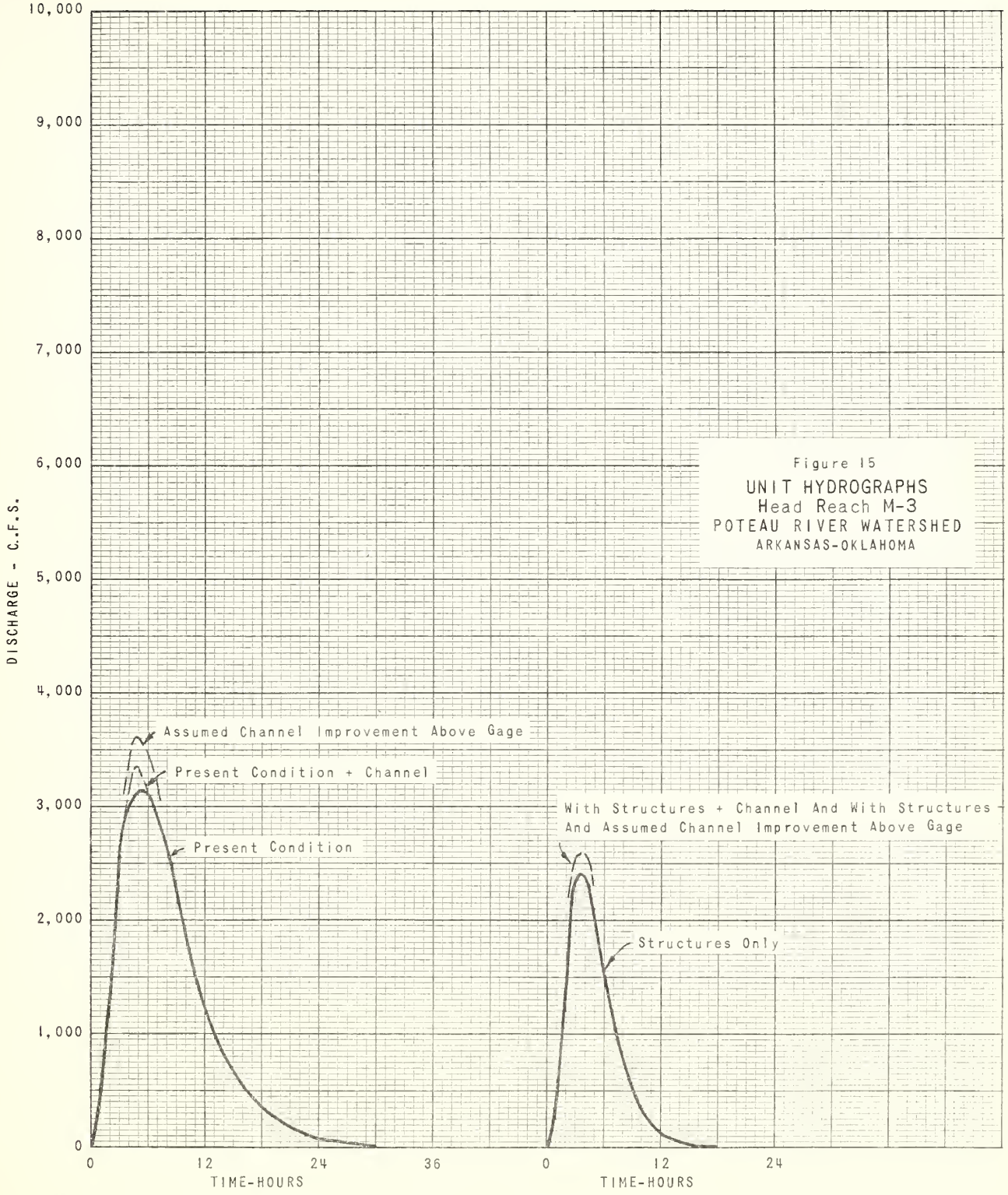

Figure 15

UNIT HYDROGRAPHS
Head Reach M-3
POTEAU RIVER WATERSHED
ARKANSAS-OKLAHOMA

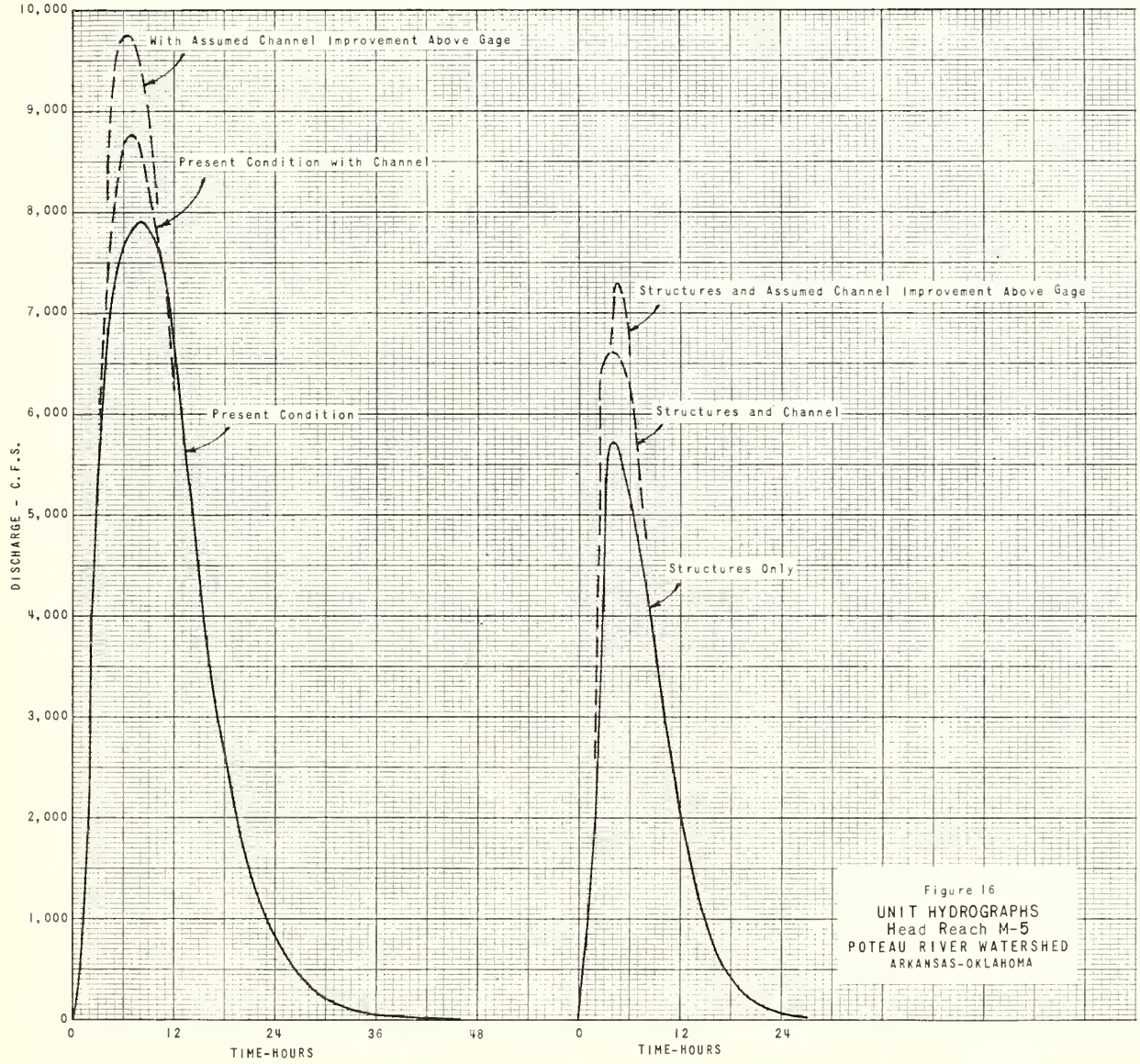

Figure 16

UNIT HYDROGRAPHS
Head Reach M-5
POTEAU RIVER WATERSHED
ARKANSAS-OKLAHOMA

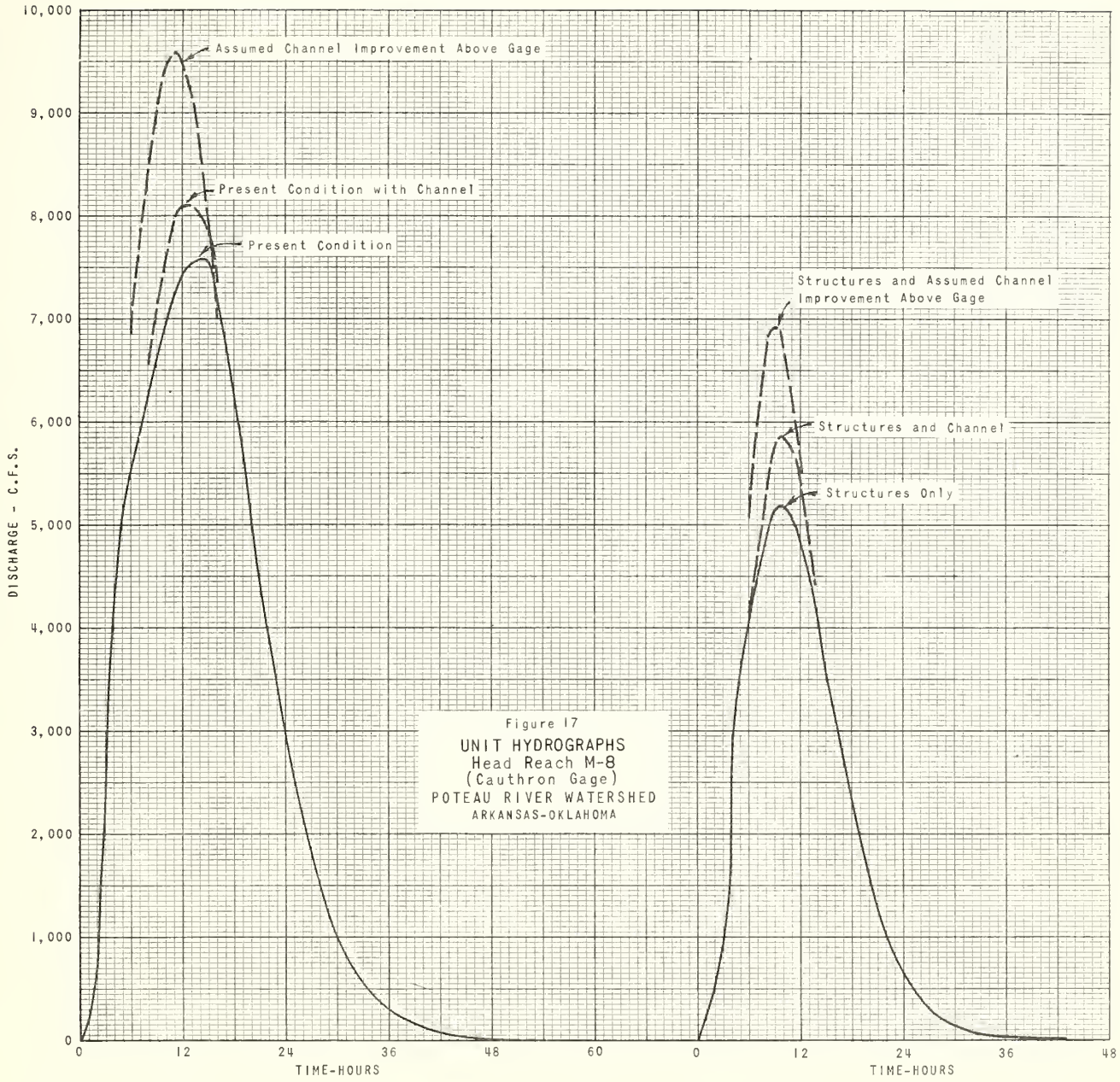

Figure 17

UNIT HYDROGRAPHS
Head Reach M-8
(Cauthron Gage)
POTEAU RIVER WATERSHED
ARKANSAS-OKLAHOMA

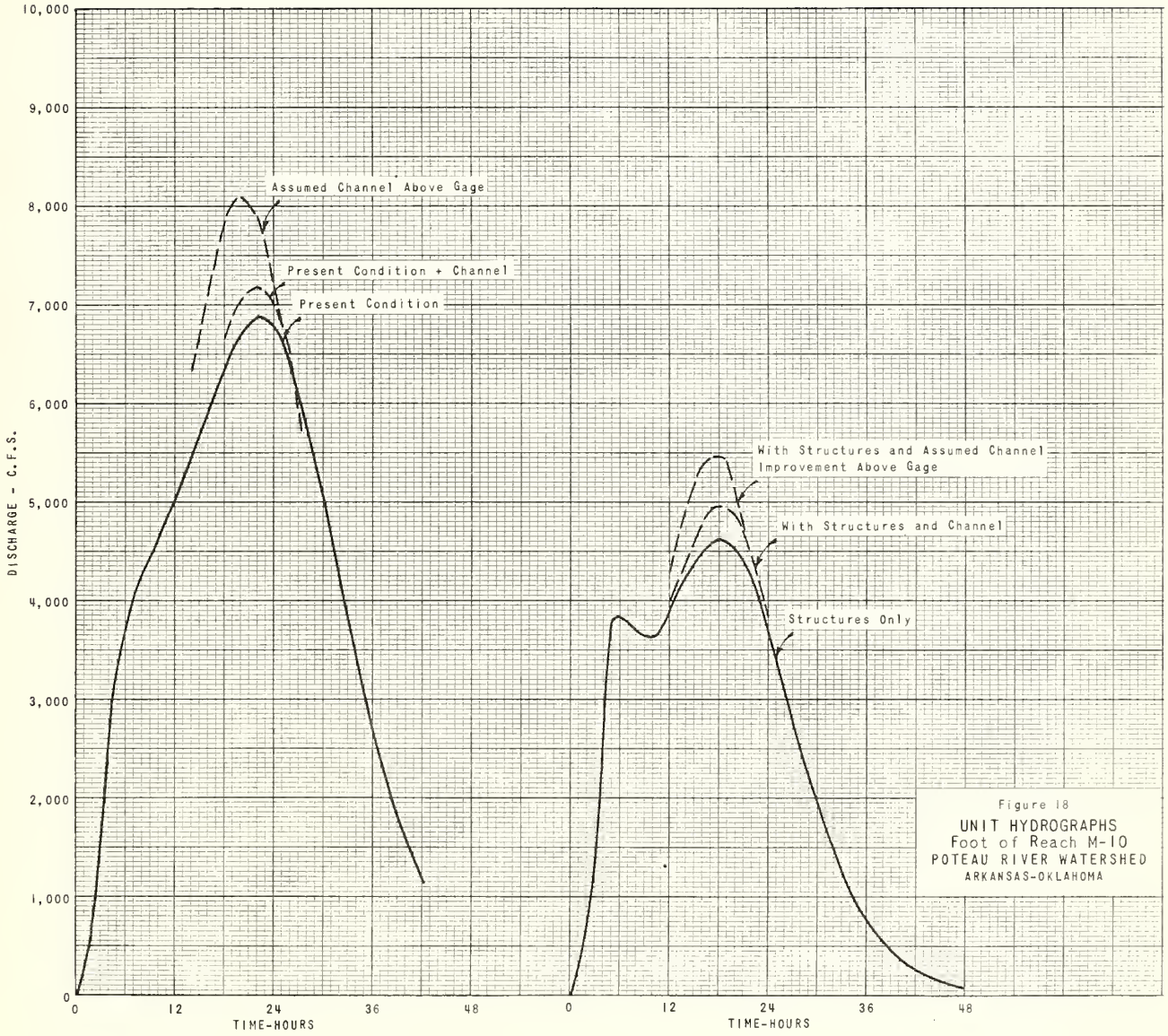

Figure 18
UNIT HYDROGRAPHS
Foot of Reach M-10
POTEAU RIVER WATERSHED
ARKANSAS-OKLAHOMA

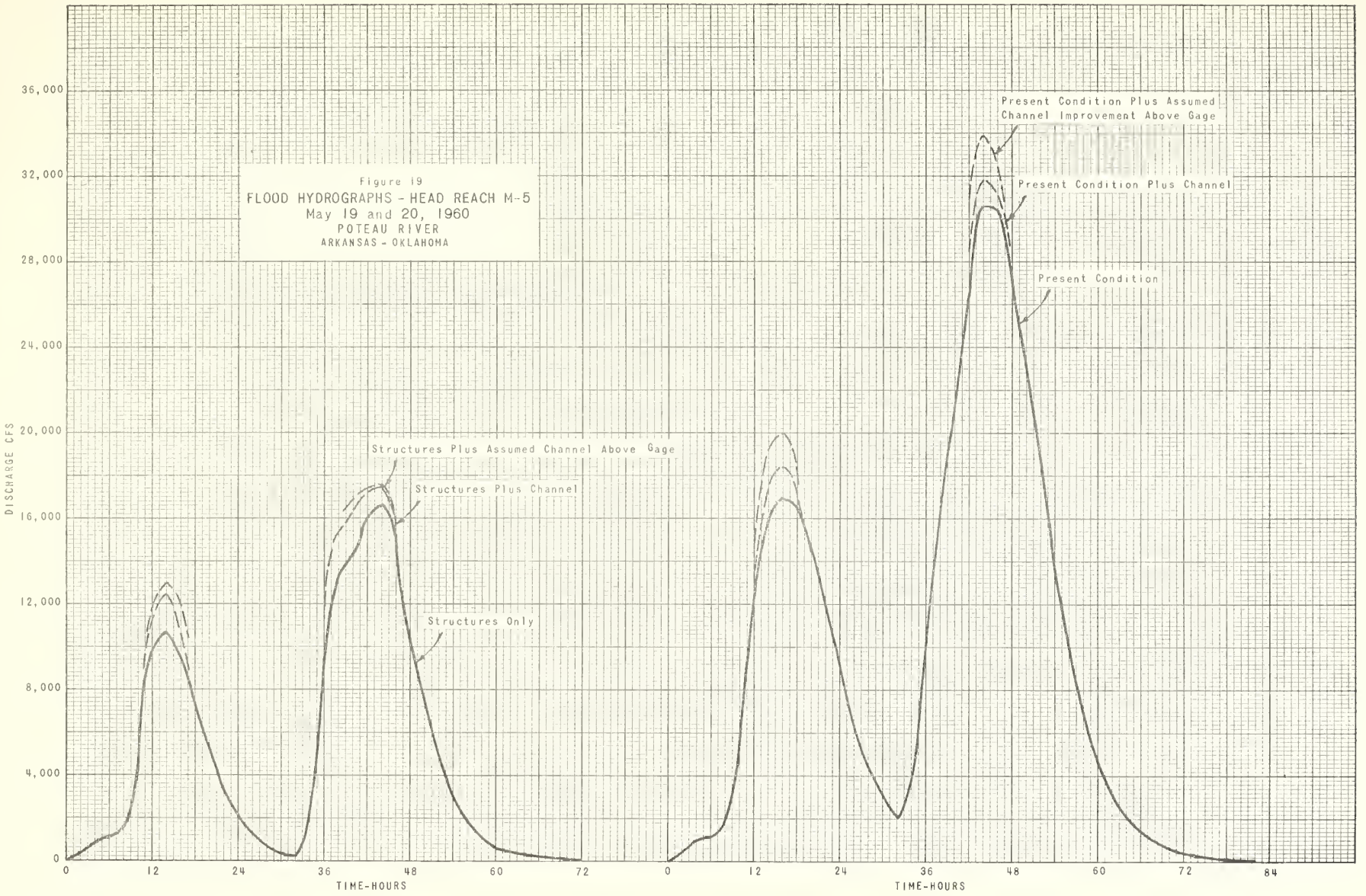
Figure 19
FLOOD HYDROGRAPHS - HEAD REACH M-5
May 19 and 20, 1960
POTEAU RIVER
ARKANSAS - OKLAHOMA
DISCHARGE CFS
36,000
32,000
28,000
24,000
20,000
16,000
12,000
8,000
4,000
0
Structures Plus Assumed Channel Above Gage
Structures Plus Channel
Structures Only
Present Condition Plus Assumed Channel Improvement Above Gage
Present Condition Plus Channel
Present Condition
0
12
24
36
48
60
72
0
12
24
36
48
60
72
84
TIME-HOURS
TIME-HOURS

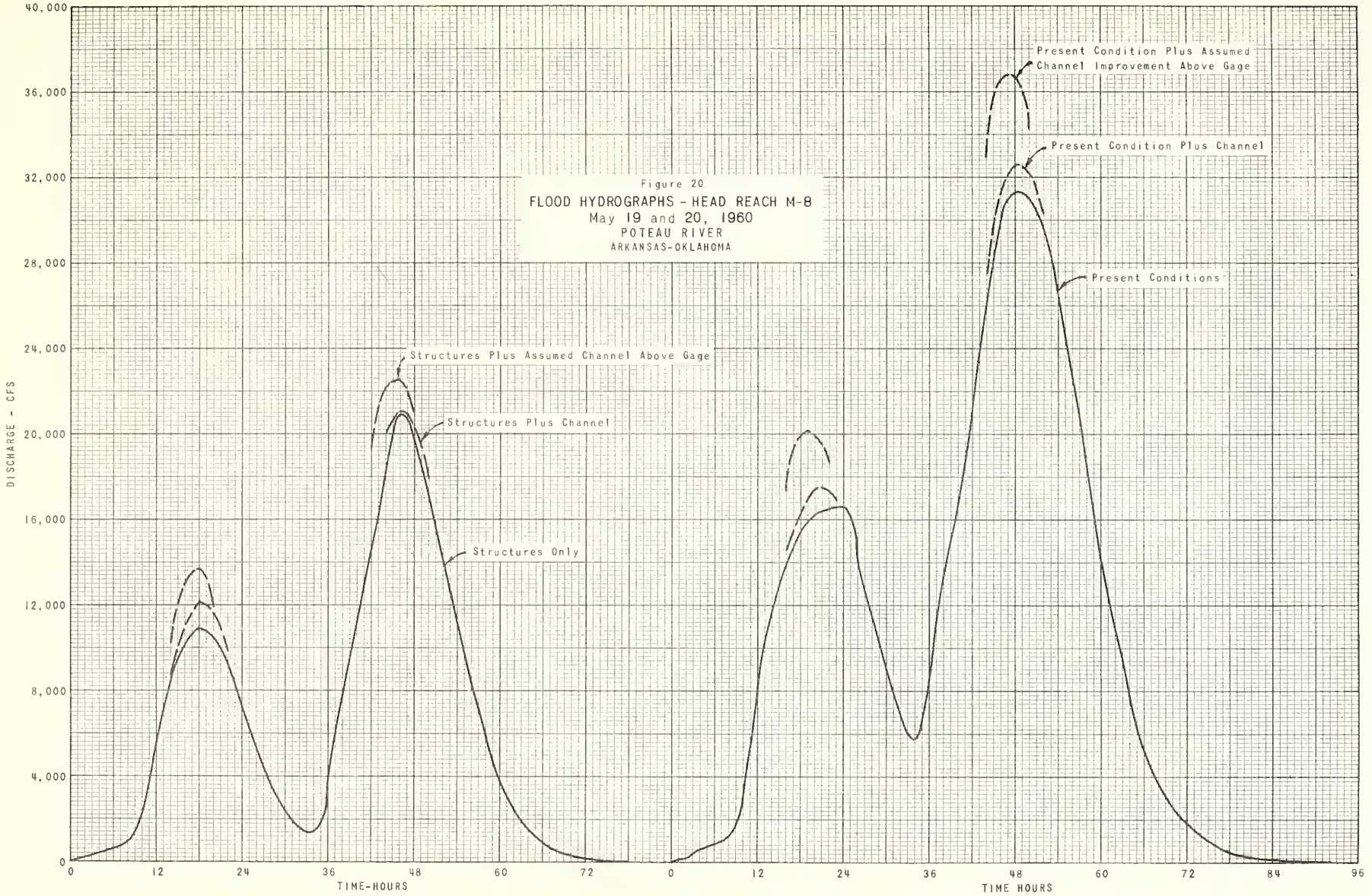
Figure 20
FLOOD HYDROGRAPHS - HEAD REACH M-8
May 19 and 20, 1960
POTEAU RIVER
ARKANSAS-OKLAHOMA
Present Condition Plus Assumed Channel Improvement Above Gage
Present Condition Plus Channel
Present Conditions
Structures Plus Assumed Channel Above Gage
Structures Plus Channel
Structures Only
DISCHARGE - CFS
40,000
36,000
32,000
28,000
24,000
20,000
16,000
12,000
8,000
4,000
0
0 12 24 36 48 60 72
TIME-HOURS
0 12 24 36 48 60 72 84 96
TIME HOURS

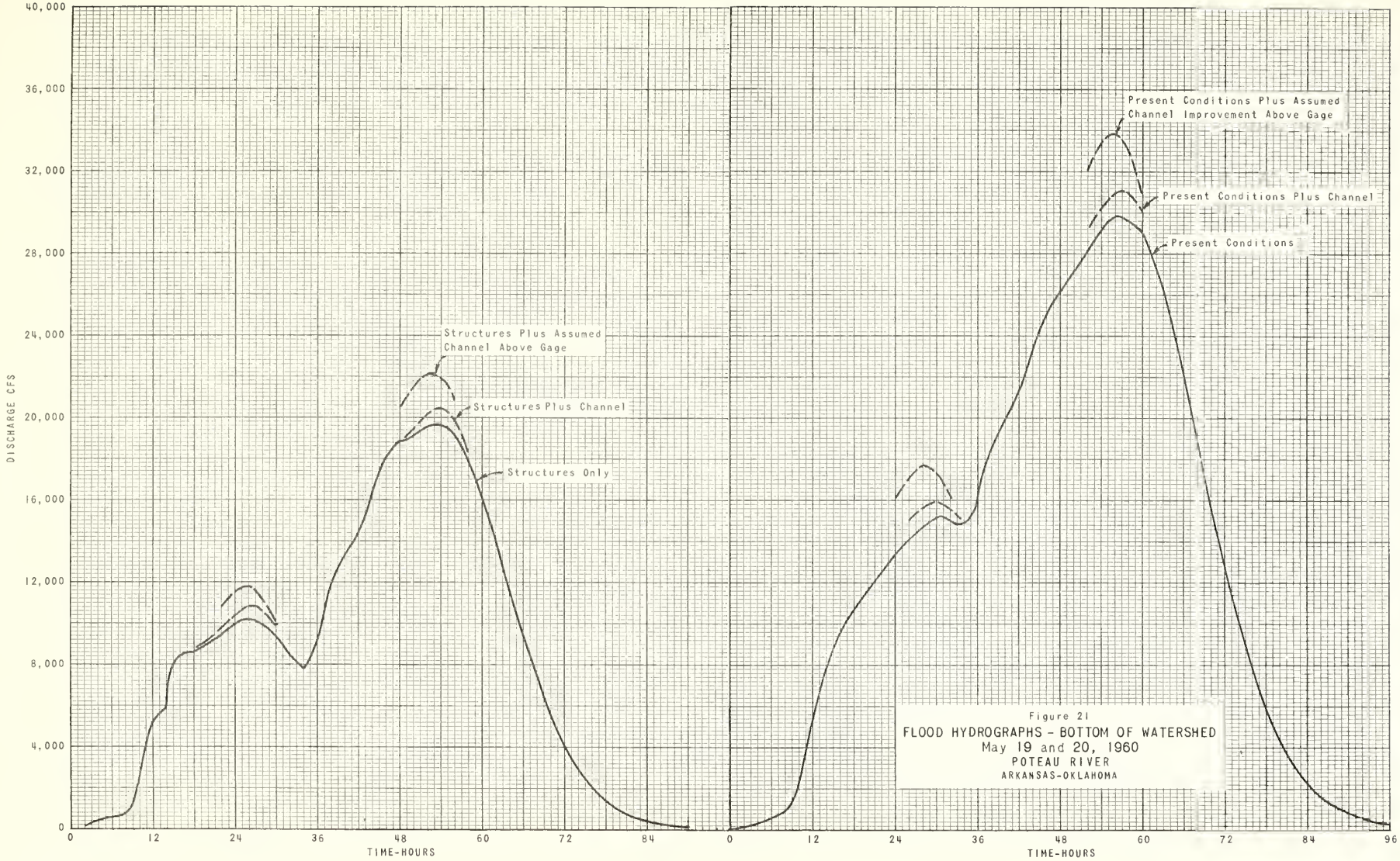

Figure 21

FLOOD HYDROGRAPHS - BOTTOM OF WATERSHED

May 19 and 20, 1960

POTEAU RIVER

ARKANSAS-OKLAHOMA

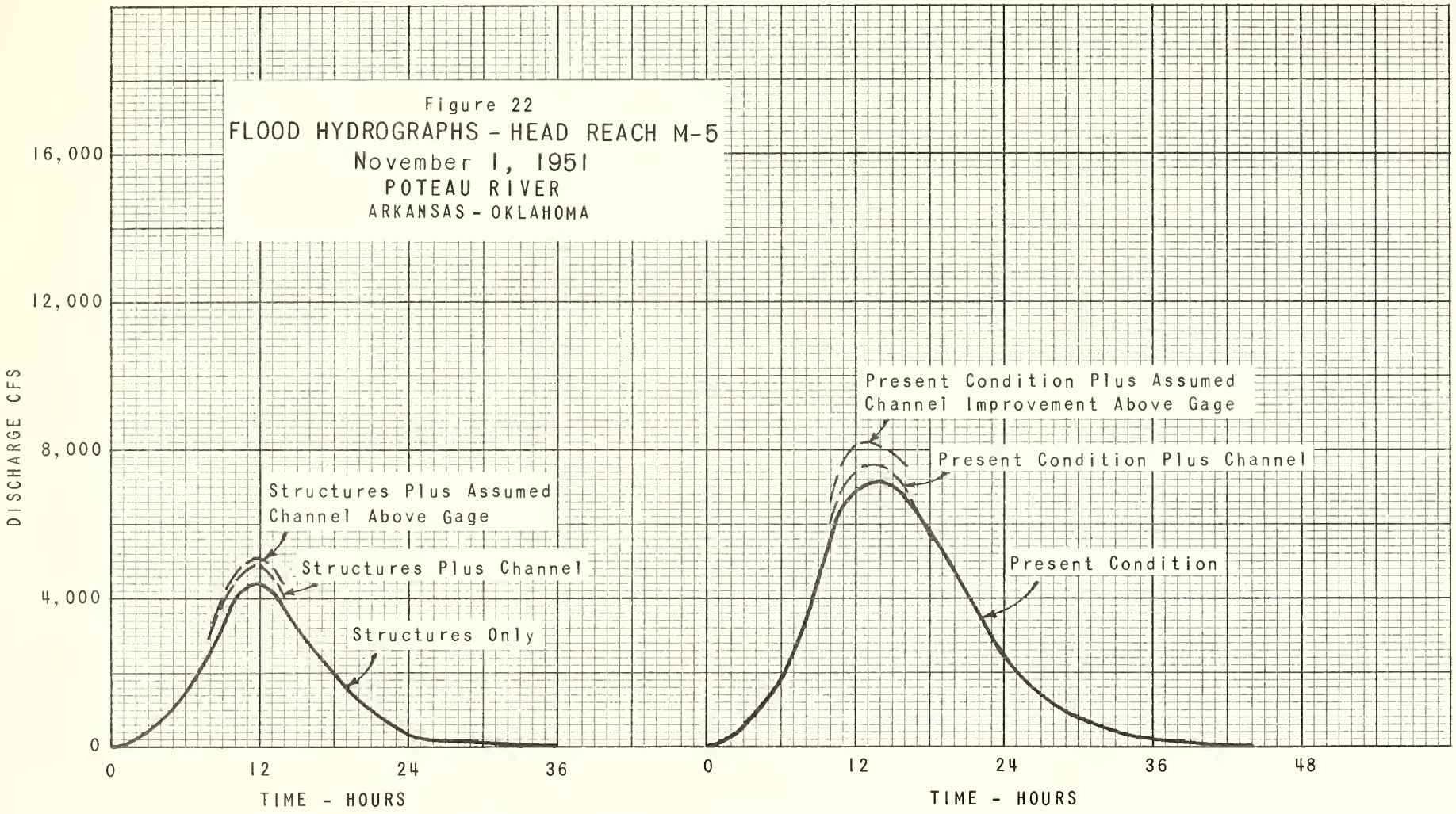
Figure 22
FLOOD HYDROGRAPHS - HEAD REACH M-5
November 1, 1951
POTEAU RIVER
ARKANSAS - OKLAHOMA
DISCHARGE CFS
16,000
12,000
8,000
4,000
0
Structures Plus Assumed Channel Above Gage
Structures Plus Channel
Structures Only
Present Condition Plus Assumed Channel Improvement Above Gage
Present Condition Plus Channel
Present Condition
0
12
24
36
48
TIME - HOURS
TIME - HOURS

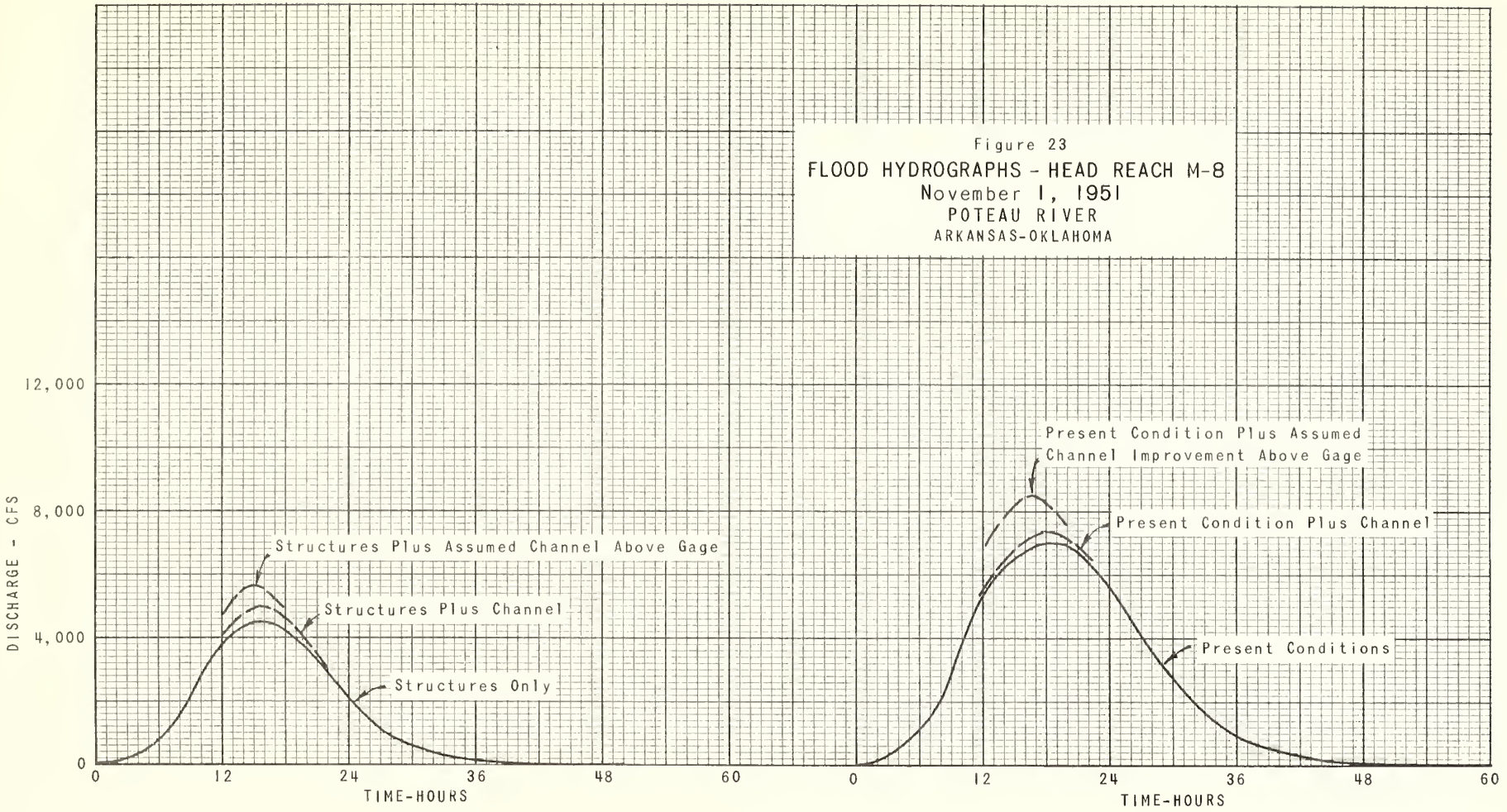
Figure 23
FLOOD HYDROGRAPHS - HEAD REACH M-8
November 1, 1951
POTEAU RIVER
ARKANSAS-OKLAHOMA
DISCHARGE - CFS
12,000
8,000
4,000
0
Structures Plus Assumed Channel Above Gage
Structures Plus Channel
Structures Only
0
12
24
36
48
60
TIME-HOURS
Present Condition Plus Assumed
Channel Improvement Above Gage
Present Condition Plus Channel
Present Conditions
0
12
24
36
48
60
TIME-HOURS

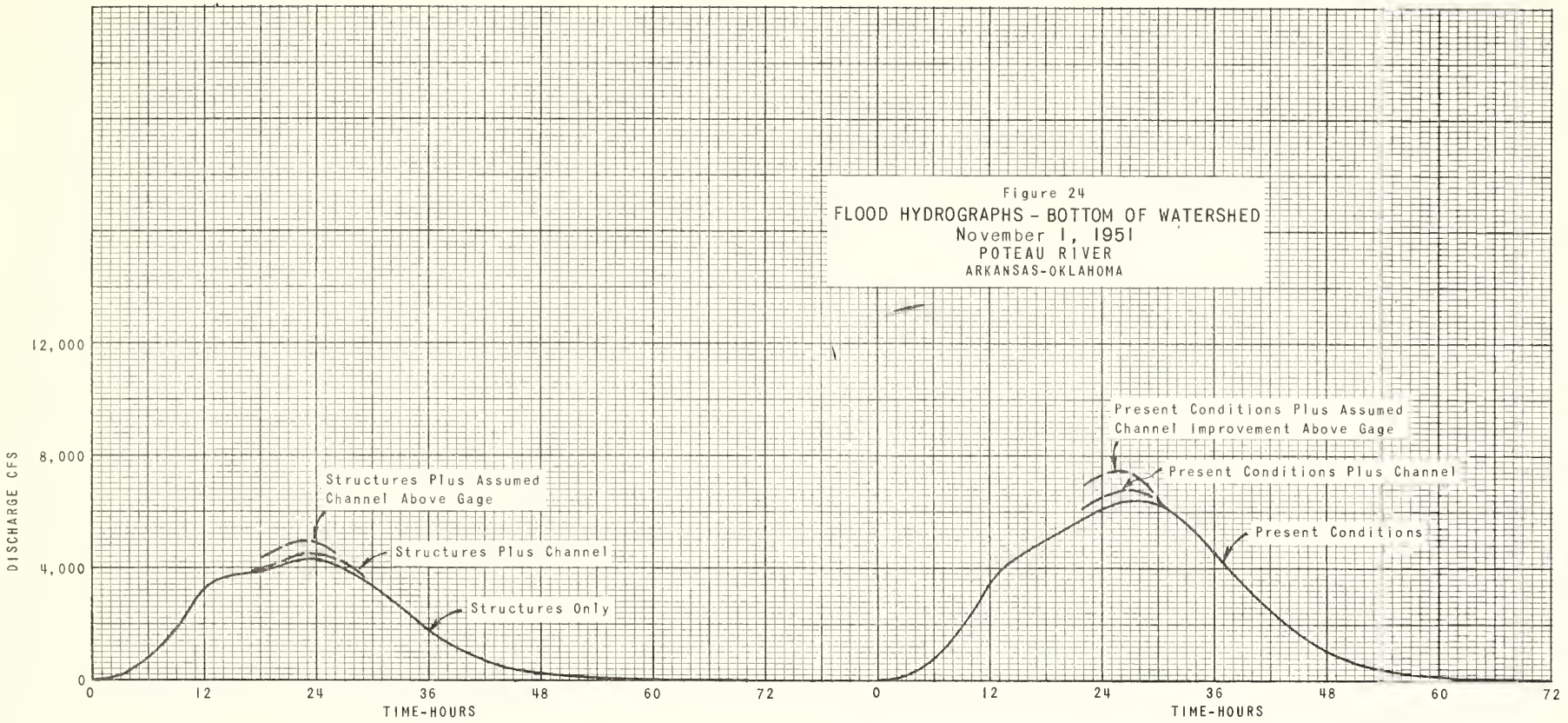
Figure 24
FLOOD HYDROGRAPHS - BOTTOM OF WATERSHED
November 1, 1951
POTEAU RIVER
ARKANSAS-OKLAHOMA
DISCHARGE CFS
12,000
8,000
4,000
0
Structures Plus Assumed Channel Above Gage
Structures Plus Channel
Structures Only
Present Conditions Plus Assumed Channel Improvement Above Gage
Present Conditions Plus Channel
Present Conditions
0
12
24
36
48
60
72
TIME-HOURS

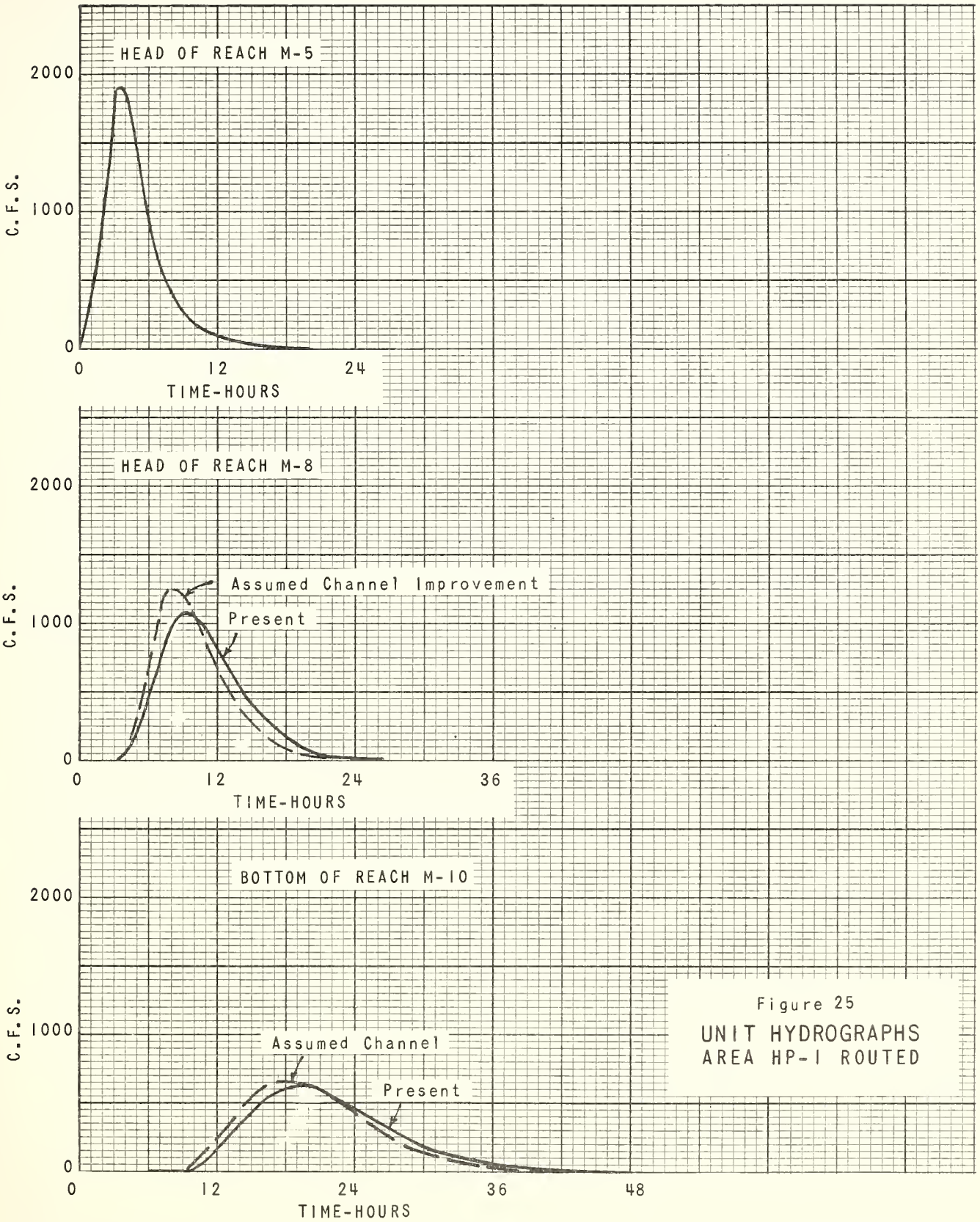

Figure 25

UNIT HYDROGRAPHS
AREA HP-1 ROUTED

Figure 26 shows, for this particular watershed, the following:

1. The percent increase in peak discharge due to channel improvement is related to the miles of improved channel in one continuous segment.
2. The percent increase in peak discharge for channel improvement with structures is greater than for channel improvement alone.
3. The increase in peak discharge caused by channel improvement with structures dissipates faster than with channel improvement alone.
4. Channel storage alone will not completely offset the increase in peak discharge caused by channel improvement.

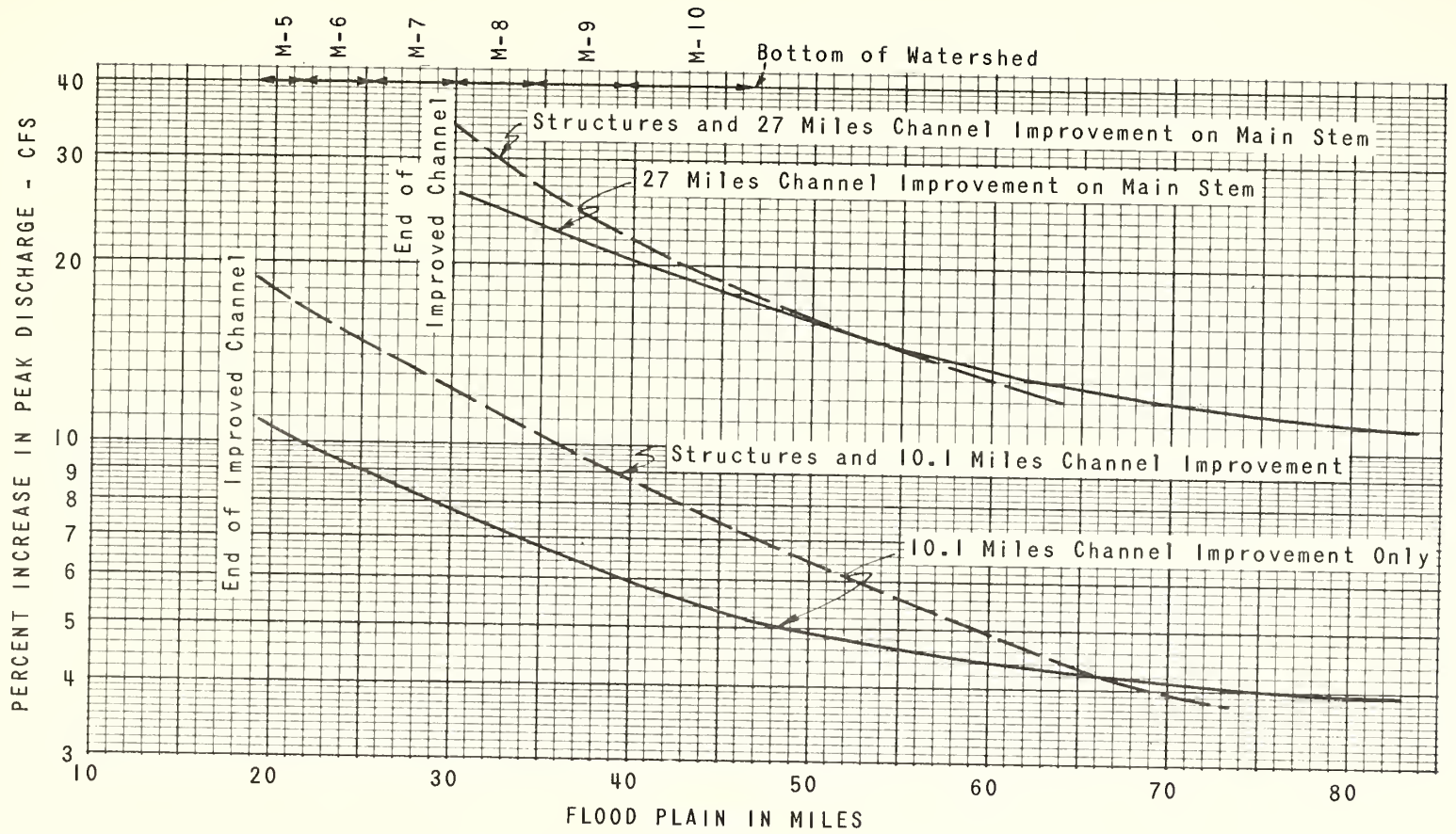

Figure 26

PEAK DISCHARGE DISSIPATION
BELOW IMPROVED CHANNEL
POTEAU RIVER
ARKANSAS - OKLAHOMA

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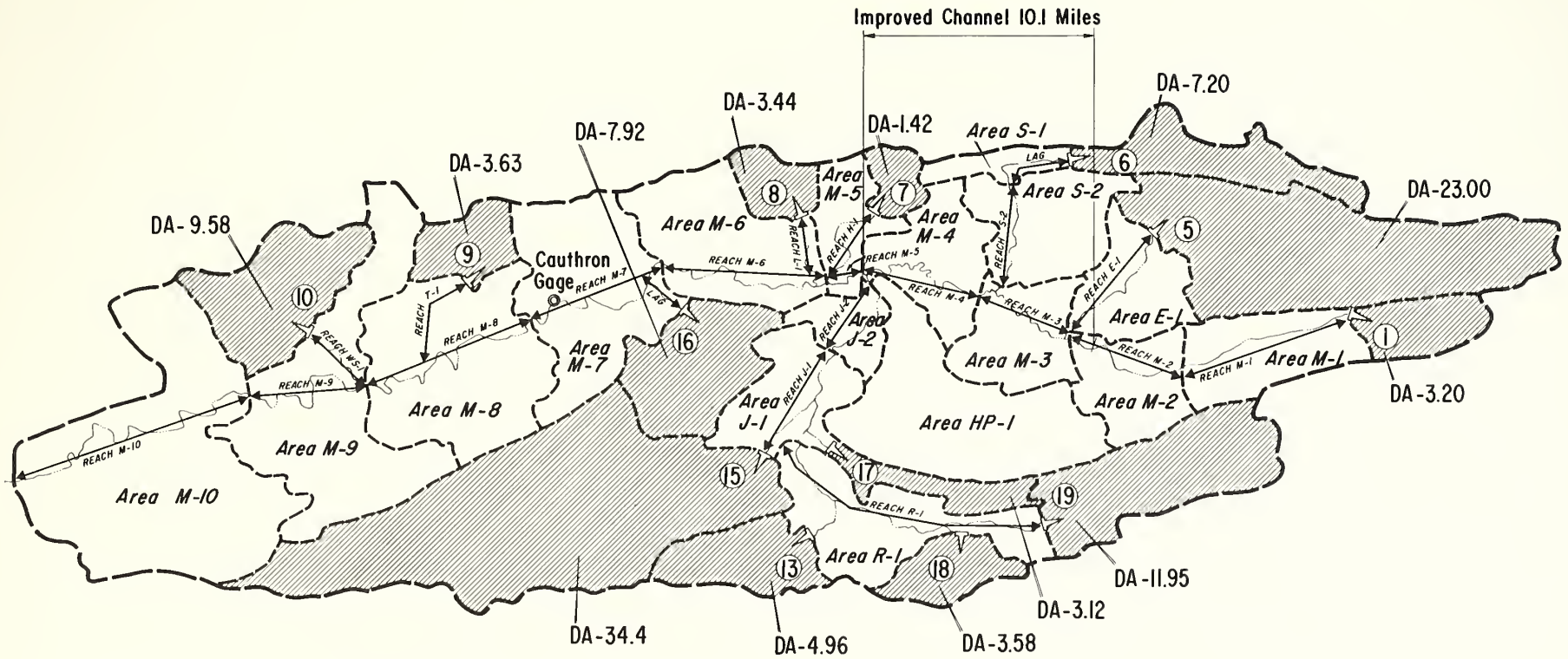

CONTROL AND ROUTING REACHES
POTEAU RIVER WATERSHED

ARKANSAS - OKLAHOMA

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